172-1760

THE LATE PROFESSOR JOHANN BAUSCHINGER
(See Appendix A.

Frontispiece.

THE

MATERIALS OF CONSTRUCTION.

A TREATISE FOR ENGINEERS

ON THE

STRENGTH OF ENGINEERING MATERIALS.

BY

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FIRST EDITION.

FIRST THOUSAND.

NEW YORK:

JOHN WILEY & SONS.

LONDON: CHAPMAN & HALL, LIMITED.

1897.

ROBERT DRUMMOND, ELECTROTYPER AND PRINTER, NEW YORK.

PREFACE.

THE rational designing of any kind of construction involves a knowledge of—

The external forces to be resisted, transformed, or transmitted;

The internal stresses resulting therefrom;

The mechanical properties of the materials to be employed to accomplish the objects sought.

Of these three coördinate departments of knowledge the first two are founded on the sciences of mathematics and applied mechanics. The last one, however, does not rest on any deductive science, as this information can only be gained by patient, expensive, and competent research. For this reason the third essential named above has not kept pace with the other two kinds of engineering science; but, on the other hand, it furnishes very much greater rewards to the skilled investigator.

During the past twenty-five years the number of such investigators has increased from a scattering few to hundreds and even thousands, and these are now found in all enlightened nations. The results of their original studies and experiments are pouring in upon us from all countries, in many languages; and no practising engineer can hope to even scan, much less to appropriate and assimilate, more than a very small part of this vast wealth of experimental knowledge.* In the following work the author presents to his readers a condensed and concise summary of such portions of the knowledge now available on this subject as he has found suitable for such a work. He is fully aware of its incompleteness and of its more or less fragmentary character. Yet with all its faults he believes it contains sufficient reliable information, not commonly accessible elsewhere, to justify its publication in this form.

* The author has one list of original contributions to the subject of the Strength of Materials filling 140 quarto pages.

When the work is used as a text-book in schools of engineering the instructor would do well to assign only such portions of Part I as are required to supplement the student's course in applied mechanics; to have his students *read* Part II if they do not get this information in other ways; to dwell longer and with more care on Part III; and to call attention only to such portions of Part IV as pertains to the particular course the students are taking. In this way the book may be made intelligent and familiar to the student, and so become to him a great and lasting aid in designing, testing, and inspecting, without requiring more time than can be devoted to the subject. This course should precede or accompany an experimental course in the testing laboratory, with which all American schools of engineering are now equipped.

An unusual use has been made of stress-diagrams and other forms of graphical representation of facts and laws, no pains or expense having been spared in this direction. So far as possible tables have been omitted and the original tabular data have been incorporated in diagrams. A law of relationship cannot be perceived from data arranged in a tabular form. When plotted to significant arguments the law not only becomes evident at a glance, but when once impressed on the mind through the sense of sight it cannot well be forgotten. To obtain this lasting benefit, however, the diagram must be intelligently read and understood. The reader is urged, therefore, to give great care to the study of all the diagrams which accompany the text on any subject, for, as a rule, the facts, laws, and conclusions to be drawn from them are not fully expressed in the text. The diagrams must be considered as a part of the text, and they should be read with even greater care than is bestowed on the word-embodied ideas.

Throughout the book, with few exceptions, both in the diagrams and in the text, the English units of weight and measure (pound and inch) have been employed. The author is of the opinion that until the metric system has been definitely adopted it is best to use the old units, and that a double system of units is confusing. The revising of books to put them in harmony with the decimal system will be but a very small part of the total expense entailed by the formal adoption of that system by our Government. As a very large part of the data given in the diagrams comes from Continental sources, all of which were expressed in the metric system, a great amount of labor was required to bring this material into the English system of units. Even the results obtained from English sources were generally expressed in long tons per square inch, so that this also required reduction to bring it to pounds per square inch.

Some of the author's usages may be regarded as unwarranted innovations. Especially may this be the case in the matter of the new elastic limit, which he proposes for general adoption, and which is discussed in Arts. 13, 261, 262, and 263. The author bespeaks for these articles a careful consideration and also a study of the many stress-diagrams scattered through the book, before his views are condemned. The fact is, something must be done

in this matter, as now no one knows what is meant by "elastic limit" without an explanation—which explanation is not usually given.

The relatively large space given to the subject of timber is not more than its importance as a structural material, and the general absence of scientific information on the subject would seem to demand. Probably the reason little has been given on this subject hitherto, in such works as this, is because little has been known. Until the Forestry Division of the U. S. Agricultural Department began the systematic study of timber and timber-trees, some five or six years ago, very little accurate or scientific information was obtainable as to the mechanical and other properties of American timber. The author's intimate connection with these investigations is a further reason why he should here present an adequate account of the work done to date.*

It has been no part of the author's aim to give working rules for using materials in structures of various kinds, or to propose original specifications to be used in the purchase of materials. He has tried to impart a knowledge of the properties of materials; on what these depend; the ordinary causes of variation and defects, and how these should be discovered; thus making the reader competent to draw his own specifications and to make his own rules.

The latest forms of investigation of metals and building-stones by means of the microscope are briefly treated (the former in Appendix B); and a chapter has been given on the magnetic properties of iron and steel, and the methods to be employed in determining these. This chapter need be read by electrical students only.

The author has acknowledged his sources of information in the text, and especially in the legends accompanying the cuts. In addition to these he desires to make a special acknowledgment here of his obligation to Professors Bauschinger, Tetmajer, and Martens; to the French Commission Report; and to Mr. Henry M. Howe, Prof. Thomas Turner, Mr. G. R. Redgrave, Prof. J. O. Arnold, Mr. Thos. Andrews, Mr. H. H. Campbell, and to Dr. B. E. Fernow. His thanks are also due to Dr. Wm. Trelease for assistance in obtaining illustrations of American trees in Chap. XIII; to Mr. H. A. Wheeler, E.M., for the chapter on the manufacture of paving-brick; to Mr. W. A. Layman, M.S., for the chapter on the magnetic properties of iron and steel; and to Prof. H. Aug. Hunicke, E.M., for revising the manuscript of Chapters VII to XI inclusive.

There are to-day a few exceptionally fertile sources of exact information on subjects pertaining to the materials of construction, prominent among which may be named:

1. The annual publications of the Results of Tests made at the U. S. Arsenal, Watertown, Mass., beginning in 1882.

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* The author has had entire charge of the mechanical tests, some 40,000 of which have been made in his laboratory at St. Louis.

2. Bauschinger's Communications from the Laboratory of the Technical School at Munich, Germany.

3. Tetmajer's Communications from the Laboratory of the University at Zurich, Switzerland.

4. Martens' Communications from the Laboratory of the University of Berlin, Germany.

5. The Report of the French Commission (of 115 members) on the Standardization of Tests of the Materials of Construction, in four quarto volumes, 1895.

6. The Monthly Journal, *Baumaterialenkunde*, published in Zurich, as the organ of the International Society for the Standardization of the Tests of Materials of Construction.

The entire engineering profession is so indebted to the late Prof. Johann Bauschinger for the work he has done in developing the scientific testing of materials that the author of this work has chosen to express his feeling of gratitude to him by using his portrait as a frontispiece and giving a brief account of his life in Appendix A.

That this work may contribute somewhat towards more rational, safe, and economic practices in the designing of all kinds of construction has been the purpose and is now the hope of

THE AUTHOR.

ST. LOUIS, MO., *Jan.* 1897.

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THE
MATERIALS OF CONSTRUCTION.

PART I.

*SYNOPSIS OF THE PRINCIPLES OF MECHANICS UNDERLYING THE LAWS OF THE STRENGTH OF MATERIALS.**

CHAPTER I.

GENERAL NATURE OF DEFORMATION AND STRESS.

1. Elastic and Plastic Bodies.—An elastic body is one which, when deformed under the application of an external force, will recover its original dimensions when the deforming force has been removed. A plastic body is one which will not recover its original dimensions after deformation. A body which will fully recover its original dimensions after deformation is said to be perfectly elastic. When it will only partially recover its original dimensions after deformation it is said to be partially elastic, and to the extent of its failure to recover its original form it may be said to be plastic.

All solid bodies are nearly or quite perfectly elastic up to a certain limit of deformation, beyond which they become partly elastic and partly plastic. This limit within which the body is nearly or quite perfectly elastic is called the *elastic limit*. When deformed beyond this limit the body will recover a portion of such deformation, the rest remaining as a permanent change or *set*. Beyond the elastic limit, therefore, a body may be said to be partly elastic and partly plastic. Practically all the materials used in engineering design may be said to be perfectly elastic within certain limits; and as these elastic limits are well beyond the limit of maximum loading in

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* This Part is intended to be supplementary to the matter contained in text-books on applied mechanics, rather than to replace such courses.

practice, it is customary to regard all engineering materials as perfectly elastic for all practical purposes.

2. Stress and Deformation.—The deformation which a solid body suffers on the application of an external force has commonly been called *strain*, but in this work it will be designated *deformation* simply.* The deformation which is fully recoverable on the removal of the external force may be called the elastic deformation. That which remains as a permanent set after the external force has been removed may be called the plastic deformation. Within the elastic limit the deformation is wholly elastic.

The relative deformation is the proportionate distortion, or the linear change divided by the original length or dimension in the direction of the deforming force. Thus if a bar 10 inches long be stretched 0.01 inch, then this 0.01 inch is the deformation, and the relative deformation is 0.01 divided by 10, or 0.001 inch. Thus the actual deformation is a concrete quantity, and is measured in units of length, while the relative deformation is an abstract number, and may be defined as the ratio of the distortion to the original length. This relative or proportionate deformation may also be defined as the deformation per unit of length.

Stress may be defined as the *resistance* a solid body offers to the deformation produced in it by the action of an external force, and it may also be defined as the resistance to this external force directly. Under the law that action and reaction are equal, the stress must be quantitatively equal to the external force, and it may be regarded as resisting this external force, or these two may be regarded as being in equilibrium. Since the application of an external force to a solid body, however, is always accompanied by a deformation of that body, and since this deformation disappears on the removal of the external force, the internal stress in the body may be said to be developed as a resistance to this deformation, and in this sense the deformation may be regarded as the immediate cause of the stress, the ultimate cause being the external force.

3. Proportionality of Stress and Deformation Inside the Elastic Limit.—Within the limits of perfect elasticity of solid bodies the deformation is directly proportional to the external force producing that change of form; and since the internal stress is of necessity equal to the external force, we arrive at this important proposition: *Inside the elastic limit the stress is directly proportional to the deformation which accompanies it.*† This

* The word *strain* is used in common language in several other senses, so that its use in this specific scientific sense, though warranted, is of doubtful propriety. The author has so used it, however, in his previous works.

† This law was first announced by Robert Hooke in 1676, in the form of an anagram, as "The true theory of *elasticity* or *springiness* ceiiinosssttuv." Two years later the key to this anagram was given in the Latin phrase "*Ut tensio sic vis*," a free rendering of which would be, "As the extension so is the strength." This law of proportionality, therefore, between the stress and the deformation within the elastic limit is frequently referred to as Hooke's Law.

proposition may be stated in another way by saying that inside the elastic limit the stress per unit area divided by the proportional deformation is a constant for any particular solid body. Since this constant is the ratio of the deforming force to the accompanying deformation of any particular solid body, it is evidently an important function, and it has therefore been given a name. The name of this ratio is the *modulus of elasticity*. We have, therefore,

$$\textit{Modulus of elasticity} = E = \frac{\textit{stress}}{\textit{deformation}}, \quad . \; . \; . \; . \; . \quad (1)$$

wherein by *stress* is meant stress in pounds per square inch, and by *deformation* is meant a *proportionate change*, or the deformation per unit of length. Thus if an external pull, P, be applied to a bar whose cross-section is A, then the unit stress is $\frac{P}{A}$; and if the length of the bar be l, and its actual extension under the application of this external force be α, then the deformation, or proportionate distortion, would be $\frac{\alpha}{l}$, whence we should have

$$E = \frac{\frac{P}{A}}{\frac{\alpha}{l}} = \frac{Pl}{A\alpha} = \frac{pl}{\alpha}, \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (2)$$

wherein p is the stress in pounds per square inch. Thus if the external force of 60,000 pounds be applied to a bar whose length is 10 inches and whose cross-section is 2 square inches, and if the extension under this load be 0.01 inch, we should have

$$= \frac{Pl}{A\alpha} = \frac{60{,}000 \times 10}{2 \times 0.001} = 30{,}000{,}000.$$

That is to say, such a material would have a modulus of elasticity of 30,000,000; and this is about the average value of the modulus of elasticity of steel.

Example.—If steel rails be welded together at a temperature of 80° F., what will be the total tensile stress in an 80-pound rail at a temperature of 20° below zero, and what will be the compressive stress in this rail at a temperature of 140° F., the coefficient of expansion being assumed as 0.0000065 per degree F.?

In solving such a problem as this, since the length of the rail cannot change for a change of temperature, the contraction which would occur if free to move is overcome by the application of a sufficient external force coming from the surrounding bodies to prevent this contraction. In other words, an external force is developed just sufficient to stretch the body as much as it would contract under a fall of temperature, and similarly an external force is exerted to compress the body as much as it would expand under a

rise of temperature. We have then only to determine the amount of the contraction or expansion from temperature and call this the deformation produced by the application of an external force, and then by the aid of the modulus of elasticity find the amount of this external force, and divide it by the area of the cross-section of the rail, thus obtaining the internal stress in pounds per square inch. The cross-section of the rail is indicated by its weight. The weight of rails is always given in pounds per yard, and it so happens that a bar of iron or steel one inch square and 36 inches long weighs just ten pounds. This unit is called an inch-yard. Therefore an 80-pound rail has just eight inches of cross-section. With the above information the student is prepared to solve the problem. It is evident that the length need not be considered; or any length may be chosen, as, for instance, one inch, since only the proportionate change of length need be considered in either case. The answers to the problem are 156,000 pounds total stress in tension at the lower temperature and 93,600 pounds total stress in compression at the upper temperature, the stress per square inch being 19,500 pounds in tension and 11,700 pounds in compression, respectively. Since the elastic limit in both tension and compression of this grade of steel is about 45,000 pounds per square inch, it is evident that these stresses are well within these elastic limits, and hence no injury to the rail would ensue from the prevention of expansion and contraction in this manner.

4. Different Kinds of Deformation and Stress.—Under the application of suitable external forces there are commonly recognized five kinds of deformation, namely: Extension, Compression, Angular, Bending, and Twisting; and corresponding with these are five kinds of stress, namely: Tensile, Compressive, Shearing, Bending, and Tortional. The last two kinds of stress are really combinations of the other three. Thus, bending stress may be resolved into tension and compression, with or without shearing, and a tortional stress is a particular kind of shearing stress. For any particular kind of material there is a definite relation between these several deformations and their corresponding stresses. The numerical values of the ratios of these corresponding deformations and stresses are the moduli of elasticity in the several cases. It so happens, however, that the modulus of elasticity, or the ratio between the stress and the deformation in tension, is usually the same as it is in compression. Both tension and compression are called direct stresses, and hence we may in general speak of the modulus of elasticity in direct stress, and the modulus of elasticity in shearing, in cross-bending, and in torsion. Since cross-bending distortion gives rise mostly to distortion in extension and compression, and their corresponding stresses, the modulus of elasticity in cross-bending may also be said to be the same as that in direct stress.

The modulus of elasticity, therefore, which is used in tension, compression, and cross-bending, is one and the same, and is sometimes spoken of as Young's modulus. That is to say, it is the ratio between *direct stress* in

pounds per square inch and the corresponding *proportionate linear deformation.*

5. Longitudinal and Lateral Deformation under Direct Stress.—The longitudinal deformation of a solid body in the direction of the deforming force is λl, where l is the original length in this direction and λ is the proportionate deformation. Hence we may write, for Young's modulus,

$$E = \frac{\text{stress per unit area}}{\text{deformation per unit length}} = \frac{p}{\lambda}. \quad . \quad . \quad . \quad . \qquad (3)$$

It is a fact of observation that when a metal body is elongated by an external force from l to $l + \lambda l$ (inside the elastic limit), it contracts laterally about one fourth of its proportionate elongation. Hence if the original diameter were d, its diameter after stretching would be $d - \frac{\lambda}{4}d$. This ratio of lateral to longitudinal deformation, under longitudinal external forces, is called *Poisson's ratio.* It is usually taken as $\frac{1}{4}$ for all metals, but for india-rubber it is $\frac{1}{2}$. The true values of this ratio, for some of the more common materials, are: *

Glass	0.2451	Brass	0.3275
Steel	0.2686	Delta-metal	0.3399
Copper . . .	0.3270	Lead	0.4282

6. Change of Volume under Direct Stress.—If the length of the body is increased by λl, and its lateral dimensions are decreased by $\frac{1}{4}\lambda d$, the new volume for a rectangular bar having lateral dimensions of b and d would be

$$l(1 + \lambda) \, . \, b\left(1 - \frac{\lambda}{4}\right) . \, d\left(1 - \frac{\lambda}{4}\right) = lbd\left(1 + \frac{\lambda}{2}\right).\dagger$$

But the original volume was lbd, hence the change of volume is $lbd\frac{\lambda}{2}$ and the *relative* change is $lbd\frac{\lambda}{2}$ divided by the original volume $= \frac{\lambda}{2}$, or the volume has been increased by one half as great a percentage as the length was increased.

If we should now apply an equal direct tension in the direction of b, we would increase this dimension by λb, and the volume by $\frac{\lambda}{2}lbd$, and similarly for a tensile force in the direction of d. Hence for a direct tensile force in

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*Taken from Wertheim and given in the Report of the French *Commission des Méthodes d'Essai des Matériaux de Construction*, 1895, vol. III. p. 6.

†Since λ is very small as compared to unity. The product of $(l + m)(l + n)(l + p)$, etc., where m, n, and p are very small fractions, is $l + (m + n + p)$, since the products of the auxiliary terms can be neglected.

all three planes the volume would be increased by $\frac{3}{2}\lambda$ times its original volume, and each dimension by $\frac{1}{2}\lambda$ times its original measure.

For a compressive force in all directions the volume would be diminished to $(1 - \frac{3}{2}\lambda)$ times its original volume, and each lineal dimension to $(1 - \frac{1}{2}\lambda)$ times its original measure.

The volumetric change of a solid body for an equal stress applied in all directions is therefore $\frac{3}{2}$ of the change of the dimension in the direction of an equal simple longitudinal stress. Thus the longitudinal proportionate deformation for a direct stress of p pounds per square inch is λ, or

$$E = \frac{p}{\lambda}.$$

But since the relative volumetric change for stress in all directions is $\frac{3}{2}\lambda$, we have as the ratio between volumetric stress and deformation under an equal stress in all directions, as a fluid pressure for instance,

$$E_v = \frac{p}{\frac{3}{2}\lambda},$$

whence

$$E_v = \tfrac{2}{3}E. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

That is to say, *the volumetric modulus of elasticity of a solid body for an equal stress in all directions is $\frac{2}{3}$ of Young's modulus, which applies only to direct stress in one plane and its accompanying deformation.**

7. Angular Deformation under Direct Stress.—We will here consider one case only of angular deformation under direct stress, and that is for equal direct stresses of opposite signs on planes at right angles to each other, as shown in Fig. 1. If the original length of each side of this cube be l, then the dimension in the direction of l_1 will be increased as much by the action of the vertical compression V as it will be diminished by the action of the horizontal tension H, since $V = H$ in pounds per square inch. Also the cube will be shortened in the direction l_2 by an amount λl due to the force

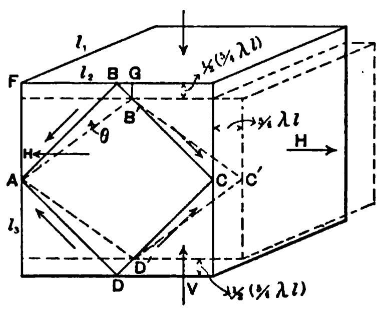


FIG. 1.

* This statement applies only to bodies in which Poisson's ratio is ¼. Since this ratio is very nearly ½ for india-rubber, it follows that the cross-section is reduced as much as the length is increased, under a tensile stress in one plane, and hence the volume remains unchanged. Similarly, for a compressive stress in all directions the volume is unchanged (almost); so that while Young's modulus of elasticity for this material is very small, the volumetric modulus is very great: and if Poisson's ratio were quite ½, the volumetric modulus would be infinite, or it would be quite incompressible. It is probably the most incompressible of any known substance.

l', and by $\frac{1}{4}$ this amount due to the lateral force H. Also the dimension in the direction l_1 will be elongated by λl from the action of the horizontal force H, and by $\frac{1}{4}$ this amount from the force V. Hence the final dimensions in these directions will be $l(1 - \frac{5}{4}\lambda)$ and $l(1 + \frac{5}{4}\lambda)$ respectively.

If in the front face of this cube the lines $ABCD$ be drawn, joining the middle points of the edges before deformation, this figure is a square. After deformation, if we make the point at A common to the two figures, we have the points B, C, and D moved to B', C', and D' respectively. This produces an angular movement of one of these lines equal to the angle BAB', which we will call θ. This is now one half the deviation of the angles $B'AD'$, $B'C'D'$, $AB'C'$, and $AD'C'$ from right angles.

But since $BG = GB' = \frac{1}{2}(\frac{5}{4}\lambda l)$, we may assume that B' falls on BC, since θ is very small.

Also,

$$\tan\theta = \frac{BB'}{AB} = \text{(from similar triangles)}\ \frac{BG}{AF} = \frac{\frac{1}{2}(\frac{5}{4}\lambda l)}{\frac{1}{2}l} = \frac{5}{4}\lambda.$$

Or, since θ is small, we may say,

$\theta = \frac{5}{4}\lambda$, where θ is given as arc in terms of the radius as unity.

But $\theta = \frac{1}{2}$ the deviation of the angles $AB'C'$, $B'C'D'$, etc., from right angles. Hence we have

$$2\theta = \text{angular change} = 2(\tfrac{5}{4}\lambda) \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

equals twice the linear deformation.

That is to say, *two direct stresses at right angles to each other and of opposite signs produce in the plane of the stresses an angular deformation equal to twice the proportionate linear deformation.* This result will be used in Art. 9 in obtaining the ratio of the modulus of elasticity in shearing to that in direct stress.

8. Relation between Shearing and Direct Stresses.—In Fig. 2 let the square $ABCD$ represent a very small portion of a longitudinal section of a body, taken in the plane of the forces. Assume also that there are shearing forces acting on the body, which have developed at this point in this plane a shearing stress on the vertical sides equal to s_1 pounds per square inch, these forming a couple and producing a turning moment. Evidently this particle can only be held from turning in this plane by the development of an exactly equal shearing stress (or resistance) on the horizontal faces, which will give an opposing couple and moment of resistance equal to the turning moment of the original shearing

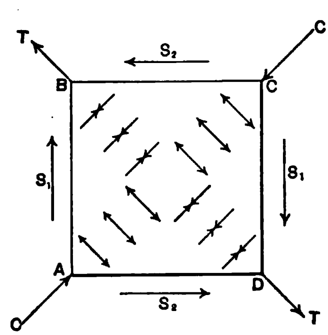


FIG. 2.

forces. If the lengths of these sides be equal, we shall then have $s_2 = s_1$ in pounds per square inch. Hence we may say:

A shearing stress in one direction at any point in a body develops an equal opposing shearing stress at right angles to it in the plane of the resultant external forces.

But the two sets of shearing forces indicated in the figure will tend to deform the body by elongating it in the direction BD, and shortening it in the direction AC. The internal resistanee to such a deformation develops in the body a direct tensile stress or resistance along the line AC and a compressive stress along the line BD.

If $S_1 = S_2$ represent the total shearing stresses on the vertical and horizontal sides of this particle, respectively (s_1 and s_2 being equal intensities of stress, or stress in pounds per square inch), then we may resolve these along the diagonals and obtain

$$\text{total tensile stress on } AC = \sqrt{S_1^2 + S_2^2} = \sqrt{s_1^2\overline{AB}^2 + s_2^2\overline{BC}^2}$$
$$= s\sqrt{\overline{AB}^2 + \overline{BC}^2} = s\overline{AC} = T$$
$$= \text{total compressive stress on } BD = \sqrt{S_1^2 + S_2^2} = C,$$

or these two direct stresses also are equal.

But the stress per square inch is the total stress divided by the area over which it acts; hence we have for the intensities of the tensile and compressive stresses

$$t = \frac{T}{AC} = \frac{C}{BD} = c = s_1 = s_2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Hence we have the larger conclusion that

A shearing stress in one direction at any point in a body develops an equal opposing shearing stress at right angles to it in the plane of the external forces, and these opposing shearing stresses produce two opposing direct stresses acting at 45° with the shearing stresses and at right angles to each other, these tensile and compressive stresses having the same intensities, in pounds per square inch, as the original shearing stress.

9. The Shearing Modulus of Elasticity.—The modulus of elasticity in shearing may be defined as *the ratio of the shearing stress in pounds per square inch to the accompanying angular deformation.* By angular deformation is here meant the angular change, as derived in Art. 7, where 2θ is a pure ratio, being the ratio of arc to radius. From the last article we know that a shearing stress gives rise to direct stresses at right angles to each other, of opposite signs, but of equal intensities; and when such stresses act, we learned in Art. 8 that the proportionate angular change was twice the proportionate linear change when equal direct stresses were acting at right angles to each other. But when both of these stresses were acting we found the linear change to be $\frac{5}{4}\lambda$, or $\frac{5}{4}$ that due only to the deforming

force in that direction; and, as found in equation (5), $2\theta = 2(\frac{5}{4}\lambda)$, we have $2\theta = \frac{5}{2}\lambda$.

But $$E = \textit{Young's modulus of elasticity} = \frac{p}{\lambda},$$

and $$E_s = \textit{shearing modulus of elasticity} = \frac{s}{2\theta}$$

$$= \frac{\textit{shearing stress per square inch}}{\textit{angular deformation}}.$$

But we have shown, when $s = p$, $2\theta = \frac{5}{2}\lambda$; hence we have

$$E_s = \frac{s}{2\theta} = \frac{p}{\frac{5}{2}\lambda} = \frac{2}{5}\,\frac{p}{\lambda} = \tfrac{2}{5}E. \quad \text{. . . . .} \quad (7)$$

That is to say, $E_s = \frac{2}{5}E$, or *the shearing modulus of elasticity* $= \frac{2}{5}$ *of the linear or Young's modulus.**

---

* This conclusion is based on a value of Poisson's ratio of $\frac{1}{4}$. The general relation between E_s and E is $E_s = \frac{8}{2\theta} = \frac{1}{2}\,\frac{m}{m+1}E$, where m is the reciprocal of Poisson's ratio. Thus if this ratio be $\frac{1}{3}$, which it is approximately for brass and copper, then $m = 3$ and $E_s = \frac{3}{8}E$, while for india-rubber, where $m = 2$, we have $E_s = \frac{1}{3}E$. Prof. Bauschinger's tests on round bars of steel give $E_s = 13{,}600{,}000$, while for square bars of the same material he found $E_s = 11{,}500{,}000$, thus showing a failure of the theory to harmonize results on these two forms of cross-section even inside the elastic limit. See Rep. French Commission, vol. III. p. 208, for Bauschinger's results.

CHAPTER II.

MATERIALS UNDER TENSILE STRESS.

10. General Phenomena accompanying Tensile Tests.—When a body of uniform cross-section is subjected to the action of an external force which tends to pull it asunder, it is elongated in the direction of this force by a proportionate amount equal to the average force per square inch divided by its modulus of elasticity; thus

$$\lambda = \text{the proportionate elongation} = \frac{p}{E},$$

where p is the external force, or internal stress, in pounds per square inch, and E is the modulus of elasticity (Young's modulus).

At the same time its lateral dimensions are reduced by one fourth as great a percentage as that which represents the proportionate elongation, as described in Art. 5. This rate of elongation in the direction of the force, and contraction in its transverse dimensions, continues in strict proportion to the amount of the external force, until the elastic limit is reached, when both the longitudinal elongation and the transverse contraction begin to increase at a more rapid rate, until finally, with the more ductile metals, the condition of perfect plasticity or viscosity is reached, and the body elongates under a constant force, while the lateral dimensions reduce more and more, until rupture finally occurs.

If the external force or load, in pounds per square inch, be represented by vertical ordinates, and the corresponding elongations be represented by horizontal abscissæ, then the action of the specimen under test may be indicated by what is known as a stress-diagram, the vertical coordinates representing stress, and the horizontal coordinates the corresponding deformations. In Fig. 3 such stress-diagrams are shown for timber, cast iron, wrought iron, and steel. These lie on the upper side of the horizontal axis. If the same materials were to be subjected to compressive external forces, corresponding stress-diagrams might be drawn in opposite directions, that is to say, downward and to the left, as indicated in Fig. 3, below the horizontal axis.

In a complete stress-diagram of a ductile metal there are four significant points which need to be noted. These are: the *true elastic limit*, the *apparent elastic limit*, the *ultimate strength*, and the *breaking-point*.

These four significant points in a tension stress-diagram are indicated by the letters *A*, *B*, *C*, and *D* in Fig. 4, where the same diagram is drawn to widely different horizontal scales.

Thus the point *A* is the true elastic limit, or the ratio of the stress to the deformation is a constant from the origin to this point. This requires that the stress-diagram should be a perfectly straight line from *O* to *A*. Beyond the elastic limit, or above *A*, the deformation sometimes increases somewhat more rapidly than it did below *A*, and the locus then becomes somewhat curved from *A* to *B*. At *B* a very marked change occurs in the

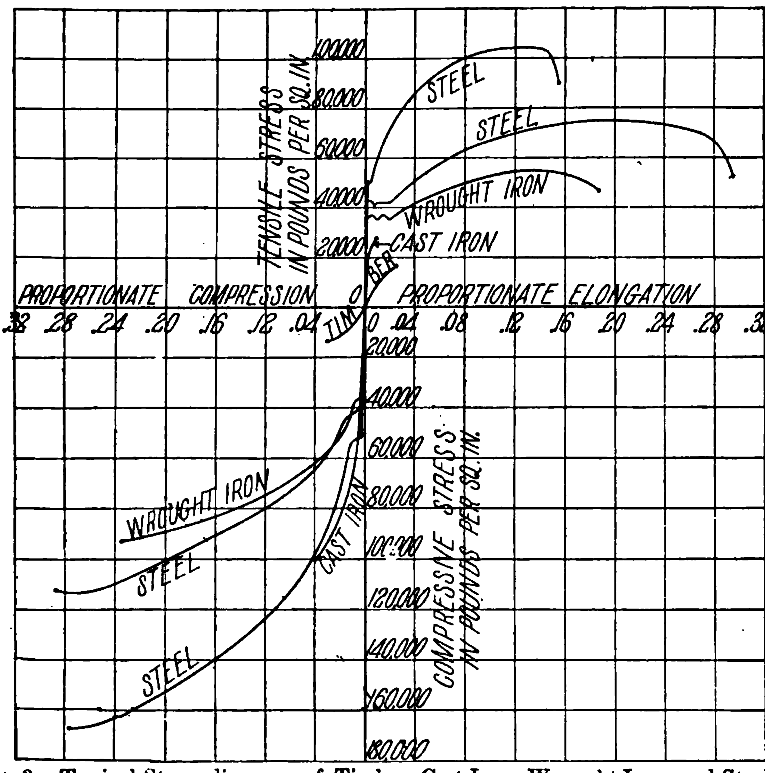


FIG. 3.—Typical Stress-diagrams of Timber, Cast Iron, Wrought Iron, and Steel in Tension and Compression, drawn to the same scales.

specimen in the case of wrought iron and structural steel. If the test be continued slowly at this point, it will be found with the more ductile metals that the specimen elongates a considerable amount under a nearly constant load, as shown in the diagram; from *B* to *B'*. This point is called the "apparent elastic limit," or the "yield-point" or the "breaking-down point." In ordinary commercial testing of iron and steel this point is always called the "elastic limit," and the true elastic limit, or the point *A*, is not found. This results from the rapid and somewhat crude methods

used in making commercial tests, and the author of this work has sometimes called this "apparent elastic limit" the "commercial elastic limit," since it is the so-called "elastic limit" found in practically all the tests made by American inspection bureaus and rolling-mills. Since this yield-point has been so long regarded as the "elastic limit," whereas the point *A* is the true elastic limit, persons who wish to be accurate and at the same

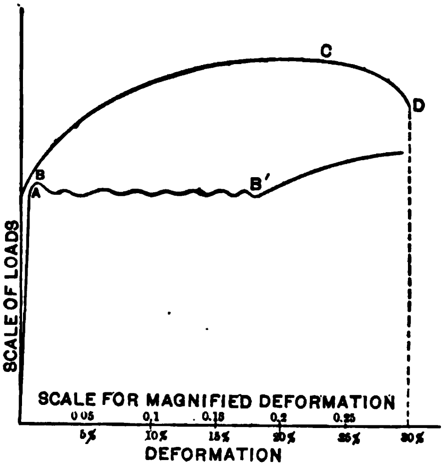


FIG. 4.

time to be understood find difficulty in conveying their meaning.* The terms "yield-point" and "breaking-down point" are not in common use, while the term "elastic limit" is commonly misused. In the present state of knowledge on the subject, therefore, the terms "true elastic limit" and "apparent elastic limit" probably would best describe the points *A* and *B* respectively.† It has been the practice of the author, in making tests to be used commercially, to call the point *B* the "elastic limit," without any explanation or exception, when he desired his results to be comparable with those made elsewhere for commercial purposes.

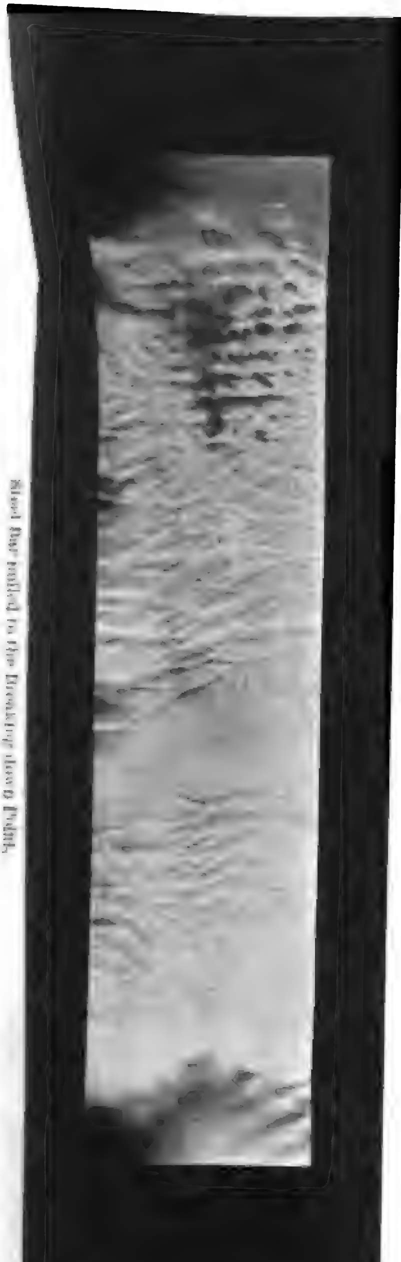
Just what happens to the specimen at the point *B* is well shown on Plate I,‡ which is a reproduction of a photograph of specimens of polished

* Fortunately, in the case of soft, or structural, steel these true points are practically identical, so that in this material no such distinction of terms as is here proposed are necessary. See Figs. 5, 6, 7, and 8.

† The French Commission use this term "apparent elastic limit" for the point *B*.

‡ The author has not seen elsewhere as clear indications of the action of such materials at the "yield-point." The tests shown on Plate I were made by him and photographed in March, 1892. The bars were polished to a mirror surface before testing. These photographs were exhibited at the Engineering headquarters at the World's Fair, Chicago, 1893, and while they were much observed and studied, it did not appear that any one had ever seen such clear "breaking-down" indications before. The significant fact is that these effects come instantly, as to any particular marking, and that they suc-

PLATE I.



Steel Bar pulled to the Breaking-down Point.

Steel Bar bent under a Uniform Bending Moment.

Photographs of a Polished Steel Bar, 1 in. × 2 in., after Bending and after Pulling, showing the "Breaking down" of the Metal.

The tensile test was interrupted before the breaking-down action had extended entirely throughout the length of the bar. (Tested and photographed by the author, 1892.)

steel subjected respectively to a uniform bending moment and to a tensile stress. This photographic reproduction shows how the tension specimen fails or "breaks down" its molecular arrangement in detail by shearing on inclined sections, beginning at the end of the specimen where it was held

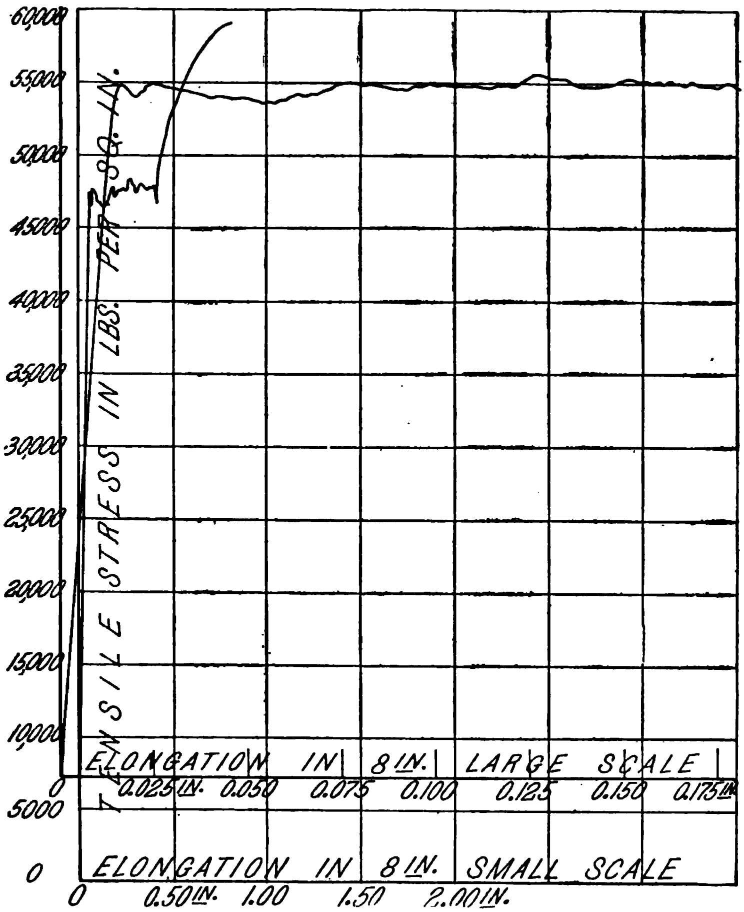


FIG. 5.—Autographic Stress-diagrams of Mild Steel, taken simultaneously with the Gray Extensometer Apparatus. Time, 2¼ minutes.

by the grips. The breaking down proceeded from the ends towards the centre. In this case the test was stopped before it had reached the middle portion. This central portion, therefore, is in its original or normal condition, while the remaining portions have been broken down in an irregular

ceed each other regularly along the bar, like the formation of ice-crystals on freezing water. The markings on the tension bar, or on the tension side of a beam, are depressions, while on a compression bar, or on the compression side of a beam, they are swellings.

weblike pattern. If the test had been continued, this action would have gone on from the ends towards the centre, until the entire specimen had yielded in this manner; and when this breaking-down action had developed over the entire length of the specimen, the point B' in the diagram would have been reached. This breaking-down action, therefore, all occurs over

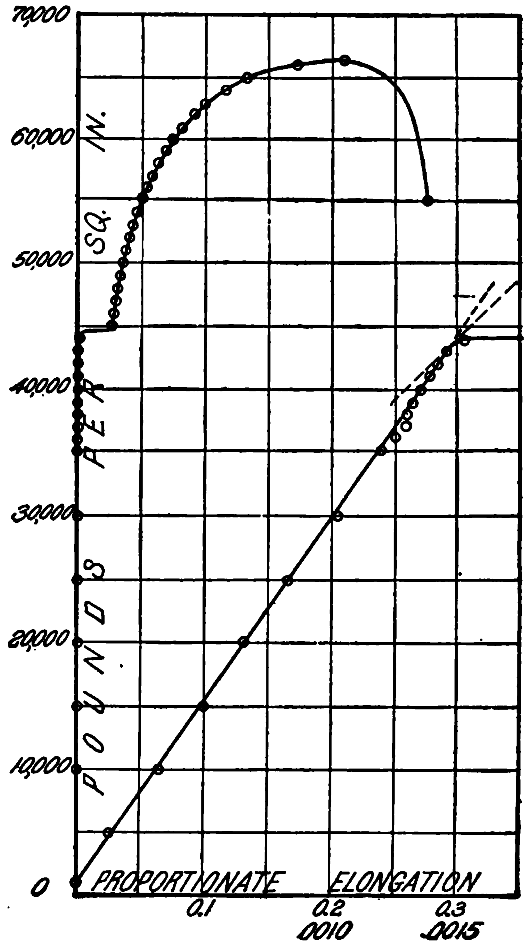


FIG. 6.—Typical Stress-diagram of Mild Steel, plotted to two scales. (From records of *Tests of Metals*, *Wat. Ars.*, 1886.)

the entire length of the specimen between the points B and B', and the reason why B stands above B' seems to be that it requires a greater force to start this breaking-down action than is necessary to continue it and extend it throughout the length of the specimen after it has once been started. See Figs. 5, 6, 7, and 8. In Figs. 7 and 8 the true elastic limit is well above the yielding resistance of the metal, or the point A is above B.

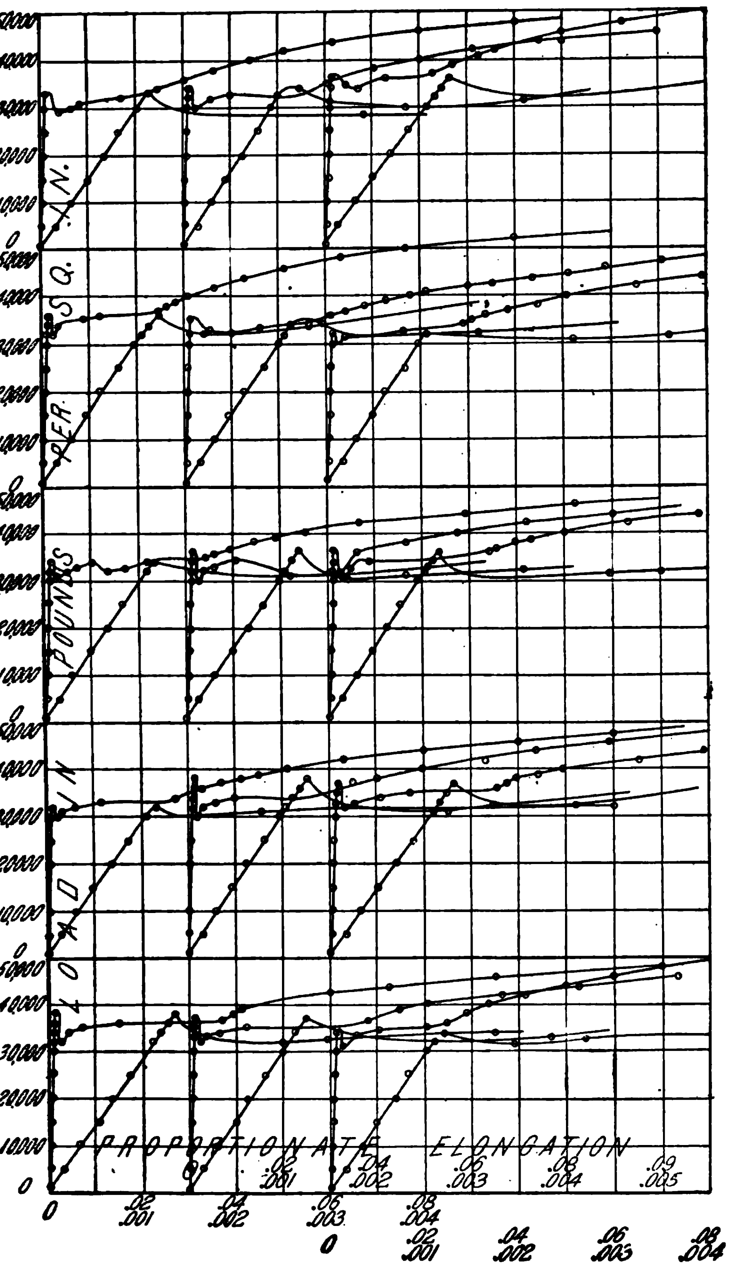


FIG. 7.—Tension Tests of Wrought-iron Shafts 1 in. in diam., used for endurance tests. Average ultimate strength = 50,400 lbs. per sq. in.; average elongation = 27% on a length of 11 diameters. (*Wat. Ars. Rep.*, 1890.)

After this breaking-down action has extended over the entire length of the specimen, a further increase in the load will continue to stretch the specimen nearly uniformly throughout its length, with a uniform reduction

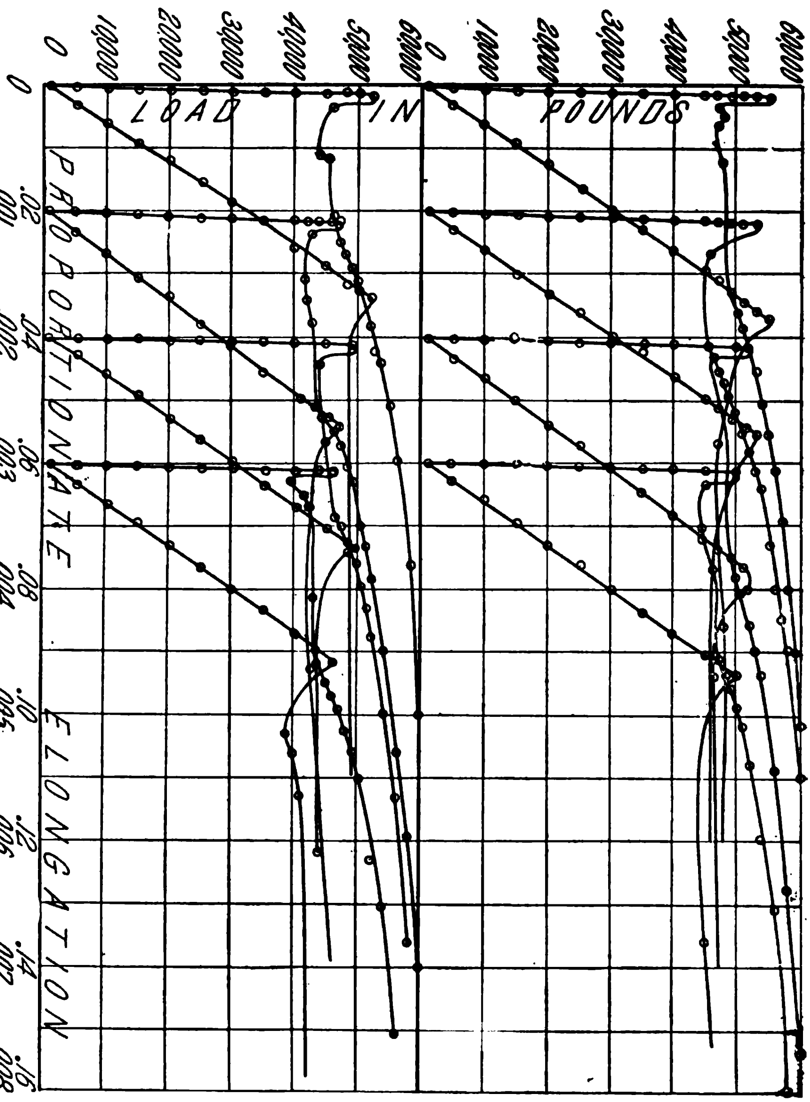


FIG. 8.—Tension Tests of Open-hearth Steel Shafts 1 in. in diameter, used for endurance tests. Ultimate strength = 61,000 lbs. per sq. in.; elongation on a length of 11 diameters = 29%. (*Watertown Ars. Rep.*, 1890.)

in cross-section, until at last the elongation and reduction continue under a constant load. That is to say, the stress-diagram becomes horizontal at the point *C*, Fig. 4. This marks the load under which the material is perfectly plastic or viscous, or for which the distortion continues with no increase of load.

After passing the point C the specimen begins to show the marked reduction of cross-section at a particular point, which will ultimately be the plane of rupture. This action is indicated in Fig. 10. As soon as this "necking-down" begins, the elongation continues under a diminishing load, as shown by the dropping of the locus in the stress-diagram, and the remaining portion of the elongation of the specimen nearly all occurs in this immediate vicinity. The area of cross-section becomes less and less, until at rupture it is perhaps less than half the original area, as shown in Fig. 10.

11. The Significant Results of a Tensile Test.—There are five significant results of a tensile test, namely:

The Modulus of Elasticity;
The Elastic Limit;
The Ultimate Strength;
The Percentage of Elongation;
The Reduction of Area of Cross-section.

The Modulus of Elasticity is found by dividing any stress per square inch below the elastic limit by the corresponding proportionate deformation. Since the stress-diagram is a straight line from the origin to the elastic-limit point, any point on this portion of the locus may be selected for the determination of the modulus of elasticity. For instance, if the point which represents an elongation of 0.1 of one per cent be chosen, the deformation being 0.001 (see Fig. 6), the modulus of elasticity is found at once by multiplying the corresponding stress in pounds per square inch by 1000.* In other words, the modulus of elasticity is the tangent of the angle which that portion of the stress-diagram below the elastic limit forms with the horizontal axis when the two coordinates are properly evaluated by the vertical and horizontal scales respectively.

It is a very remarkable fact that the modulus of elasticity of all grades of wrought iron and rolled steel, from the softest up to the highest grade of spring steel, is nearly constant, and has a value from 29,000,000 to 30,000,000, being perhaps always within the limits of 27,000,000 and 31,000,000 pounds per square inch. The ultimate strength of these metals varies from about 45,000 to several hundred thousand pounds per square inch for the strongest steel wire; but through this range of variation of strength the ratio of the stress to the corresponding deformation remains nearly constant. The modulus of elasticity is, therefore, a very valuable quality of such materials and one which is made great use of in

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* If the diagram is not straight to this point (has its elastic limit below this point), then draw a tangent to the diagram at the origin, and note where it cuts the ordinate marking a deformation of 0.001, and this stress multiplied by 1000 is the modulus of elasticity. This modulus can be read off in this manner from any of the stress diagrams for tension and compression found in this work.

engineering design. It may be called the modulus of stiffness, since it is a direct measure of the rigidity of a body, or an inverse measure of its flexibility.*

12. The True Elastic Limit is, in general, from 50 to 70 per cent of the ultimate strength of the material, while the apparent elastic limit is from 60 to 70 per cent of the ultimate strength of the material. The apparent elastic limit, or the breaking-down point, is also the ultimate strength for practical purposes, since almost all materials lose their value in structural designs after they have been deformed beyond this limit.

The true elastic limit may be defined either as the deformation where permanent set begins, or as a point beyond which a given increment of load produces a greater increment of deformation, which is the point where the ratio of the stress to the deformation ceases to be a constant and begins to diminish. This is also the upper extremity of the straight portion of the stress-diagram. If a material like wrought iron or structural steel be loaded beyond its true elastic limit, and even beyond its yield-point, and the load removed, the material has been permanently elongated; but if it again be subjected to a load, it will be found to be perfectly elastic up to the limit of its previous loading. In other words, *its elastic limit has been raised to the value of its previous loading.* In this way the elastic limit can be raised practically up to the ultimate strength. When the term "elastic limit" is used in a scientific sense without modification, the true or primitive elastic limit (point A, Fig. 4) is always to be understood; but when used in a commercial sense, the apparent elastic limit or yield-point (point B) is to be taken.

As stated previously, the elastic limit is usually found in commercial testing by noting the action of the weighing-beam in dropping under an increasing stretch, this being in fact the breaking-down point. To determine the true elastic limit it is necessary to use very delicate measuring appliances, which will enable the observer to discover when the ratio of stress to deformation has begun to change. Even when using such devices the readings must be plotted to a large scale to detect the deviation from a straight line.

13. "The Apparent Elastic Limit" is defined by the French Commission as "the load per square millimeter of the original section, where the deformation begins to increase sensibly with no increase in the external force applied (corresponding to the dropping of the beam in testing-machines)."† Since in most kinds of materials there is no such point other than the ulti-

* A modulus of flexibility would be the reciprocal of the modulus of elasticity, or $\frac{1}{E}$, but Prof. A. B. W. Kennedy has taken for such a modulus of "*specific extension*" the stretch in thousandths of an inch on a length of 10 inches under a stress of 1000 lbs. per sq. in. Its reciprocal multiplied by 10,000,000 is the modulus of elasticity.

† Report of the French Commission, vol. I. p. 207.

mate strength, and since in these materials an elastic limit corresponding to sensible deformations is required for practical purposes, *the author proposes to extend the meaning of this term so as to make it applicable to all elastic materials, and at the same time to make it serve as the "elastic limit" to be universally used in all kinds of practical tests.* For this purpose he employs the following definition:

*The apparent elastic limit is the point on the stress-diagram of any material, in any kind of test, at which the rate of deformation is fifty per cent greater than it is at the origin.**

This point is found either by comparing increments of deformation with given increments of load, or better by plotting the stress-diagram and drawing a tangent to it which has an inclination to the vertical 50 per cent greater than has the tangent to the diagram at the origin, as shown in Fig. 9. To

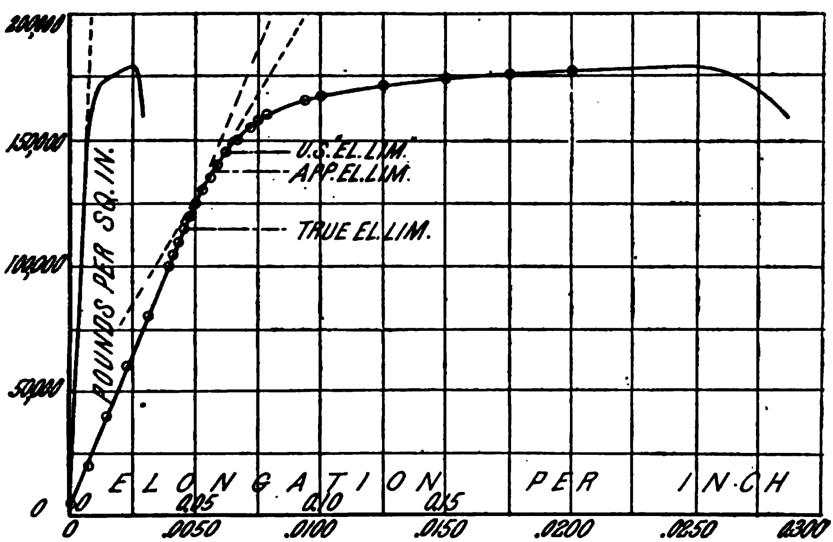


FIG. 9.—Stress-diagram of Hard-drawn Steel Wire. (*Wat. Ars. Rep.*, 1890.)

do this lay off the tangent to the curve at the origin (or inside the true elastic limit, where it is a straight line), and then fix a point on any horizontal line of the paper, 50 per cent farther from the vertical axis than the point where this tangent cuts it. Lay a parallel ruler on this point and the origin, and move it till it becomes tangent to the curve and draw the tan-

* This definition should not be made to apply, however, to materials not perfectly elastic within any limits. Thus certain stones and concretes have stress-diagrams which are reversed curves, their rates of deformation being greater at first than after they are heavily loaded, and any load produces a permanent set, as shown in Chapter XXXI. Here the modulus of elasticity is different for every increment of load, and no kind of "elastic limit" can be attributed to them. That is to say, they are not perfectly elastic for any load however small.

gent line. Then fix the point of tangency by the eye, and call this the *apparent elastic limit.**

This fixes a point which in all cases corresponds to an extremely small permanent deformation. In Fig. 9 the permanent deformation at this limit, for hard-drawn steel wire, is about 0.0003 of the length, or $\frac{3}{100}$ of one per cent, while the limit so fixed is some 22,000 lbs. per sq. in. above the true elastic limit. Although this test was made at the U. S. Arsenal at Watertown, Mass., and on the Emery testing-machine, with extreme accuracy, as shown by the accordance of the results when plotted to the large scale in Fig. 9, yet the "elastic limit" as set down in the published record (which "elastic limit" is supposed to be the "*true elastic limit*") lies some 8000 lbs. per sq. in. *higher* than the "*apparent elastic limit*" fixed by the rule here laid down! This same state of affairs is shown in numberless cases in the recorded results of tests made at this the most fruitful and accurate laboratory in the world.† While, therefore, objection would be quickly raised to the criterion herein proposed for fixing an "apparent elastic limit" in so arbitrary a manner, and apparently so far beyond the "true elastic limit," yet no one would be inclined to question the records of the U. S. Watertown Arsenal tests, in the fixing of a "true elastic limit," even though this should in nearly all cases lie beyond this conventional "apparent limit"! After a great deal of thought and research given to this subject, the author believes no better criterion can be found for fixing a practical "elastic limit" which will be one and the same limit for a given material in the hands of all experimenters, and on all machines. For all materials which have a definite "yield-point" this "apparent elastic limit," determined as here described, will agree with it exactly; but for such materials it would never be determined in this manner, since it is then so much more readily found by the "drop of the beam," or even by a pair of dividers set to given marks on the specimen. For all materials which have no point of "yielding under a fixed load" at this stage of the test, this criterion would always accomplish the following results:

1. It would always fix one and the same well-defined point.
2. This point would always correspond to so small a permanent deformation as to be, for all practical purposes, the true elastic limit.
3. It is equally applicable to all materials which have an elastic field.
4. It is equally applicable to all kinds of tests, whether on specimens or on finished members or structures, where deformations of any kind can be correctly measured.

While the 50 per cent increase in the rate of deformation is purely arbi-

* The author has done this in his U. S. timber tests since 1891, calling this point in his cross-bending stress-diagrams "the relative elastic limit."

† See other instances in records selected therefrom for this work in Chapters XXV and XXVI.

trary, it is not large enough to fix a point having an appreciable permanent set, but it is large enough to fix a well-defined point on the stress-diagram. A very extended experience in its application, therefore, serves but to confirm the author in its continued use, and in the recommendation of its general adoption which is here put forth for the first time.*

14. The Ultimate Strength of a specimen subjected to tensile stress is measured by the maximum load carried, and is indicated on the stress-diagram by the true maximum point in that curve. It is found by dividing the maximum breaking load by the original area of cross-section. In case of the more plastic metals, the area of the broken section is usually about one half the original area, so that the ultimate strength of the actual section at rupture when found by dividing the breaking load by the final area of this section would be about twice the ultimate strength as computed on the original section. That is to say, the drawing down and pulling out of the metal has nearly doubled its strength per square inch. *The term "ultimate strength," however, always refers to the original section, and is found by dividing the maximum load by the original section.*

15. The Percentage of Elongation is found by dividing the increase of length after rupture has occurred, by the original length. By original length is meant a certain portion of the specimen which has been reduced to a uniform cross-section before testing. A standard length for tensile-test specimens in America and in England is eight inches, while in Germany and France it is twenty centimeters, these standard lengths being practically identical. The elongation of a test specimen of the plastic metals may be divided into two portions: (*a*) that part of the elongation which is uniformly distributed over the section; (*b*) that part of the elongation which occurs in the vicinity of the section which finally breaks. Thus in Fig. 10 are shown four sets of test specimens of mild steel, there being three specimens in each set. All the specimens of one set were originally of the length indicated by the untested specimen which stands on the left side of each group. The specimen next adjoining it on the right has been stretched to the limit of the elongation indicated in (*a*) above, or until there is an indication of a local reduction of area. The right-hand specimen in each group shows the local elongation and reduction, but the specimen has been removed from the testing-machine before rupture occurred. The middle specimen of each group has been tested to the ultimate strength of the material, since, when the specimen begins to reduce locally, the ultimate strength has been passed, and the strain-diagram begins to fall, or it is developed under a diminishing load.

By the amount, therefore, that the right-hand specimen in each of these groups is longer than the middle specimen of the group, by so much has the length been increased by the local drawing out on the section where failure will finally occur. The first elongation, therefore, is that portion which is uniformly distributed over the specimen, and the second is that

* See also Arts. 261, 262, and 263, pages 306–311.

Fig. 10.—Showing the Necking-down Action of Steel Bars before Rupture. (Tetmajer, vol. IV.)

which is concentrated in the vicinity of the final failure. Both of these elongations are, however, measured and included in the total elongation, from which the percentage of elongation is determined. The total elongation is obtained after rupture has occurred, by placing the two ends together and measuring the distance between the primitive gauge-marks. In the case of specimens having shoulders at their ends the gauge-marks should be at least one-half inch inside of the shoulder, since the metal adjacent to the shoulder does not elongate fully, because of the strengthening effect of the enlarged cross-sections at the ends.

It will at once be apparent from a study of these specimens that the (*b*) elongation, or that which is locally developed in the vicinity of final rupture, is nearly the same in all these specimens; whereas the (*a*) elongation, or that which is uniformly distributed over the specimen, is always directly proportional to the length. The total elongation, therefore, will not be proportional to the length. In other words, the percentage of total elongation will be greater for the short specimen than for the long ones. This shows the necessity of using standard lengths of these specimens when the percentage of elongation is to be found.

The percentage of elongation is the result which indicates the ductility of the material, this being one of the most important qualities of the metals used in structural designing.

16. The Reduction of Area of Cross-section is found by determining the area of the broken cross-section, subtracting this from the original area of cross-section, and dividing the difference by the original area. This is not so important an indication or result as the others described above, but it is customary to determine it, and to add it to the record. For the ductile metals this reduction of area may be as much as from fifty to sixty per cent of the original cross-section.

CHAPTER III.

MATERIALS UNDER COMPRESSIVE STRESS.

17. Two Classes of Engineering Materials.—Engineering materials may be divided into two general classes, according to their manner of failure in compression.

Plastic or viscous materials are those which will flow without showing any other indication of failure under a sufficient compressive load.

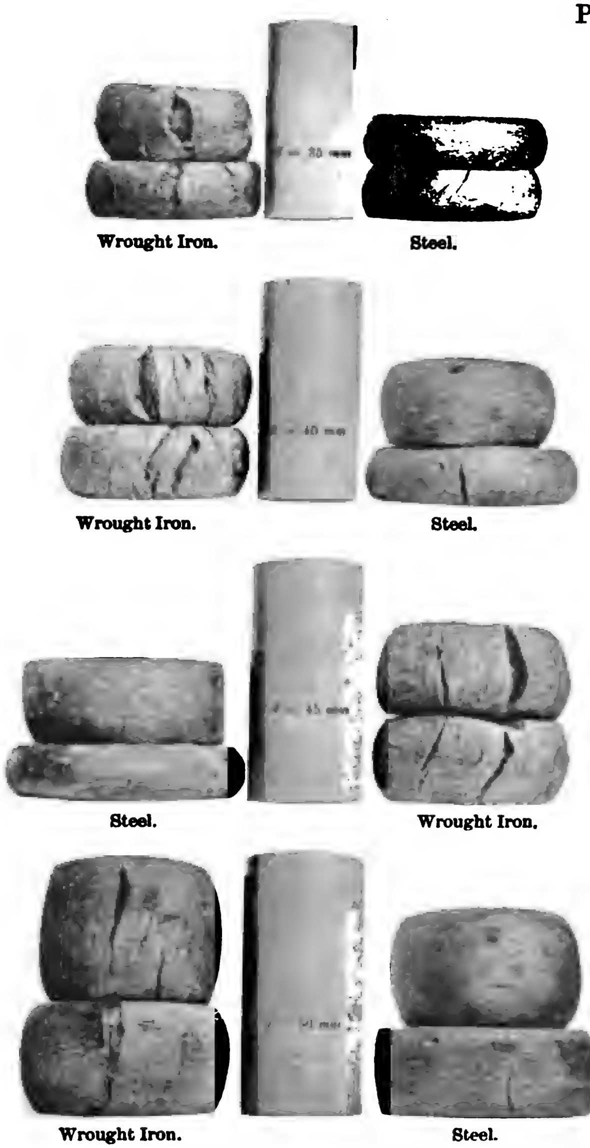
Brittle or comminuible materials are those which will crush to a powder, or crumble to pieces, or fail by shearing on definite angles under a compressive load.

In the former class are such materials as wrought iron, soft and medium steel, the alloys, lead, copper, zinc, and the like. Of the latter class are cast iron, hard or tempered steel, brick, stone, cement, etc. The laws of failure of these two classes are very different, and they will, therefore, have to be discussed separately.

18. Crushing Strength of Plastic or Viscous Materials.—There is no such thing as an "ultimate strength" in compression of a plastic body. There is, however, a definite "apparent elastic limit," the same as in tension. Beyond this limit the material simply spreads, and increases the area of its cross-section indefinitely under an increasing load, as shown in Plate II. The elastic limit in compression of such a material is the greatest load from which the specimen will fully recover, or it is the greatest load within which the stress and deformation bear a constant ratio to each other. This elastic limit in compression for wrought iron and steel is, fortunately, about the same in pounds per square inch as the elastic limit in tension. It is not customary, therefore, to test such materials in compression, but to assume that they have the same elastic limit in compression which they are found to have in tension.

19. The Law Governing the Strength in Compression of a Brittle or Comminuible Material.—Experiments show that all such materials when subjected to a compressive load fail by shearing on certain definite angles. The resistance to movement along these angles is made up of two parts: first, the strength of the material to resist shearing; and second, the frictional resistance to motion along this plane. The sum of these two resistances must equal the shearing component of the load imposed when

PLATE II.



RELATIVE MALLEABILITY OF WROUGHT IRON AND SOFT STEEL.

All the specimens were originally of the shape of the one remaining undeformed. The wrought-iron specimens uniformly show large cracks. (From von Tetmajer's *Communications*, vol. IV, Pl. V.)

resolved along the shearing plane. To find what this angle should be, we may equate the two resistances here described with the shearing force, and find the angle of rupture, the determining condition being that this angle shall be that which offers the least total resistance to failure under a crushing load. This angle may be found in the following manner:

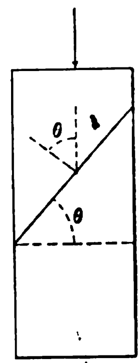


FIG. 11.

Let s = shearing strength of the material per square inch;
A = area of prism = 1 square inch;
θ = angle of rupture;
p = crushing load per square inch.

The tendency to slide on the plane of rupture is $p \sin \theta$.

The resistance to sliding is $s \sec \theta + fp \cos \theta$, where f is the coefficient of friction $= \tan \phi$, where ϕ = angle of repose. Hence, at failure,

$$p \sin \theta = s \sec \theta + fp \cos \theta. \quad \ldots \ldots \quad (1)$$

It is evident that the angle of rupture will be such as to cause failure under the least load; hence if θ be taken as the independent variable, we shall have at rupture

$$\frac{dp}{d\theta} = - s(\cos^2 \theta - \sin^2 \theta + 2f \sin \theta \cos \theta) = 0,$$

or

$$f = - \frac{\cos^2 \theta - \sin^2 \theta}{2 \sin \theta \cos \theta} = - \frac{\cos 2\theta}{\sin 2\theta} = - \cot 2\theta. \quad \ldots \quad (2)$$

Whence, since $f = \tan \phi$, we have

$$\tan \phi = - \cot 2\theta = - \tan (90° - 2\theta) = \tan (2\theta - 90°),$$

or

$$\phi = 2\theta - 90° \quad \text{and} \quad \theta = \frac{90° + \phi}{2} = 45° + \frac{\phi}{2}. \quad \ldots \quad (3)$$

That is to say, *the angle of rupture is* 45° *plus one half the angle of repose.*

If the friction had been omitted, we should have had

$$p \sin \theta = s \sec \theta; \quad \text{whence} \quad \frac{dp}{d\theta} = - s(\cos^2 \theta - \sin^2 \theta) = 0;$$

$$1 - 2 \sin^2 \theta = 0; \quad 2 \sin^2 \theta = 1, \quad \text{or} \quad \theta = 45°. \quad \ldots \quad (4)$$

It has been customary to neglect the friction, and to state that the planes of rupture make this angle of 45° with the horizontal;* but the actual plane of rupture, when the specimen has sufficient height, is about

* *Coulomb* is responsible for this theory, while *Navier* has given the true analysis. Most writers, including Rankine, have followed Coulomb, however.

55° with the horizontal, or 35° from the direction of the applied load. See Figs. 12 and 13, showing tests on sandstone made by Prof. Bauschinger.) Mr. Charles Bouton has shown* that the theoretical angle of rupture is

FIG. 12.—Bauschinger's Compression Tests on Sandstone.

borne out in practice with many kinds of materials. (See Fig. 14 for photographic views of crushed specimens of cast-iron cylinders of various heights, showing angle of rupture.)

The following table gives the results of Mr. Bouton's determinations of the theoretical and the actual values of this angle:

* In a thesis for the degree M.S. at Washington University, 1891, entitled *Theory and Experiments on the Laws of Crushing Strength of Short Prisms.* Mr. Bouton also derived the formulæ in this article and afterwards found that Navier had anticipated him.

Material.	Number of Experiments.	Observed Angle of Rupture. θ	Observed Angle of Repose. ϕ	Theoretical Angle of Rupture. $45° + \frac{\phi}{2}$	Differences.
"F" cast iron	24	54°.8 ± 0°.2	20°.6	55°.3	− 0°.5
"C. W." cast iron	24	55 .0 ± 0 .2	16 .9	53 .4	+ 1 .6
Limestone	4	62.2	33 .4	61 .7	+ 0 .5
Asphalt paving mixture	3	59.7	27 .3	58 .6	+ 1 .1
Milwaukee brick	4	58.2	27 .0	58 .5	− 0 .3

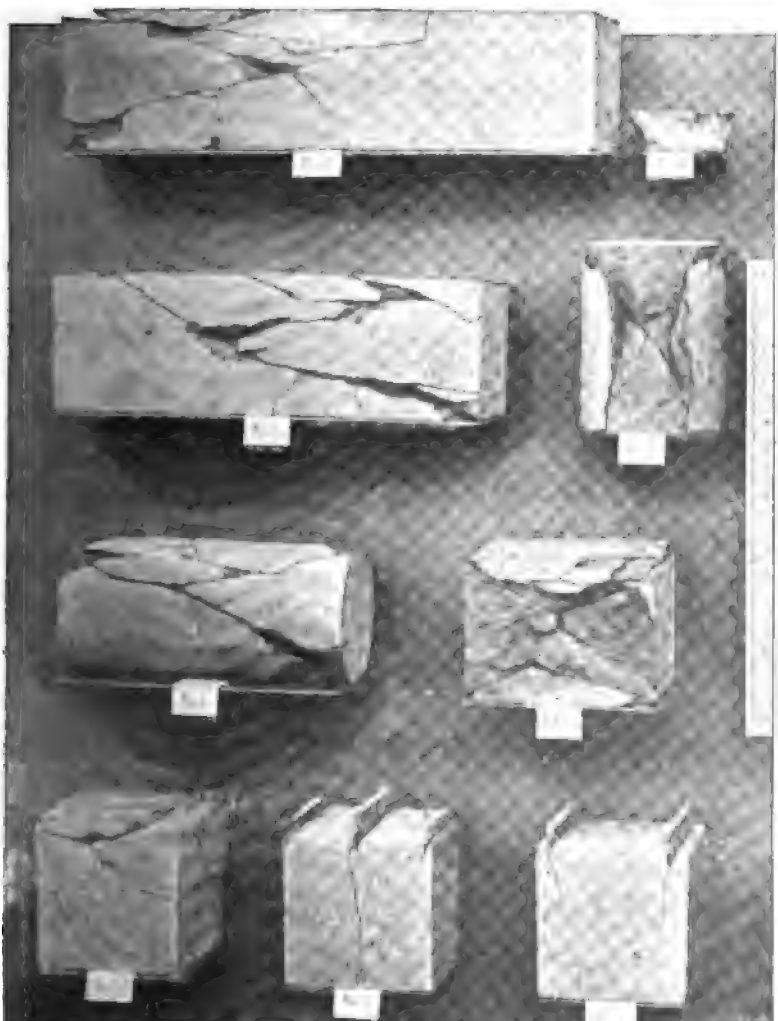


FIG. 13.—Bauschinger's Compression Tests on Sandstone.

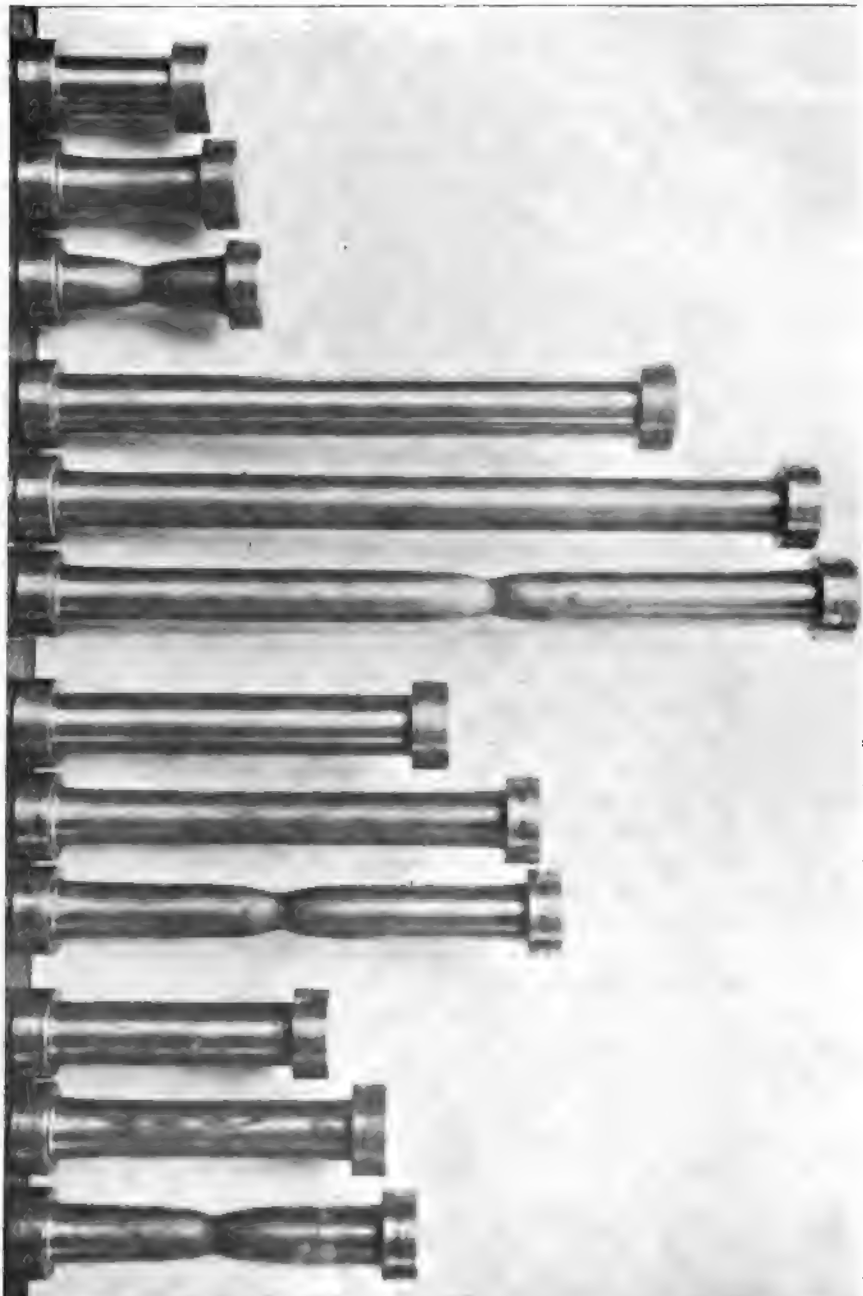


FIG. 10.—Showing the Necking-down Action of Steel Bars before Rupture. (Tetmajer, vol. IV.)

which is concentrated in the vicinity of the final failure. Both of these elongations are, however, measured and included in the total elongation, from which the percentage of elongation is determined. The total elongation is obtained after rupture has occurred, by placing the two ends together and measuring the distance between the primitive gauge-marks. In the case of specimens having shoulders at their ends the gauge-marks should be at least one-half inch inside of the shoulder, since the metal adjacent to the shoulder does not elongate fully, because of the strengthening effect of the enlarged cross-sections at the ends.

It will at once be apparent from a study of these specimens that the (*b*) elongation, or that which is locally developed in the vicinity of final rupture, is nearly the same in all these specimens; whereas the (*a*) elongation, or that which is uniformly distributed over the specimen, is always directly proportional to the length. The total elongation, therefore, will not be proportional to the length. In other words, the percentage of total elongation will be greater for the short specimen than for the long ones. This shows the necessity of using standard lengths of these specimens when the percentage of elongation is to be found.

The percentage of elongation is the result which indicates the ductility of the material, this being one of the most important qualities of the metals used in structural designing.

16. The Reduction of Area of Cross-section is found by determining the area of the broken cross-section, subtracting this from the original area of cross-section, and dividing the difference by the original area. This is not so important an indication or result as the others described above, but it is customary to determine it, and to add it to the record. For the ductile metals this reduction of area may be as much as from fifty to sixty per cent of the original cross-section.

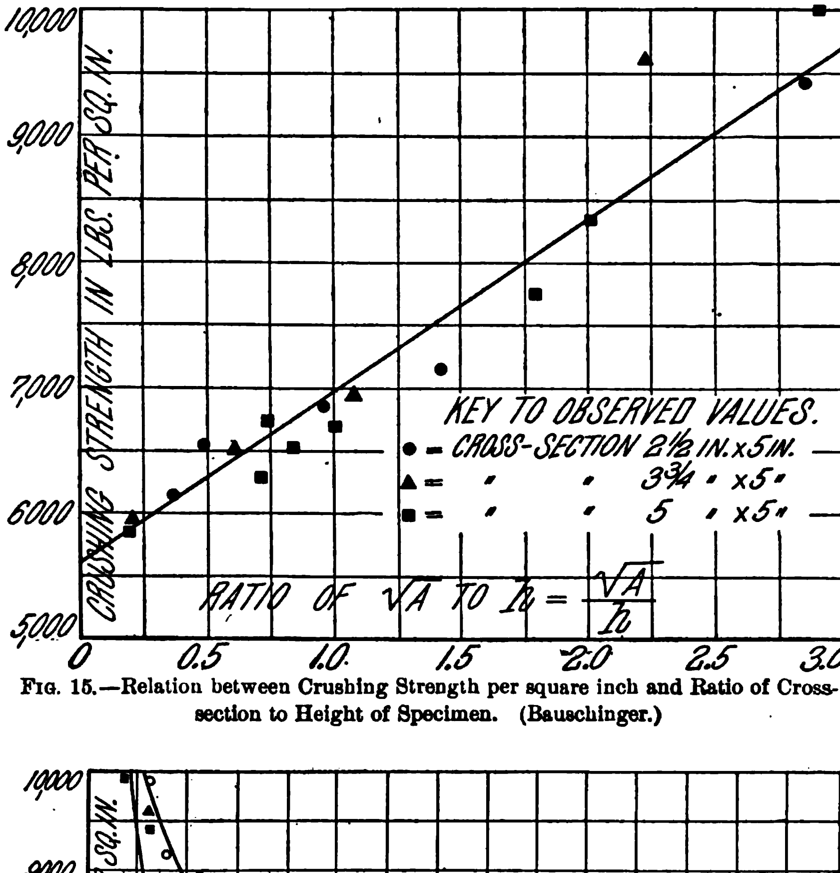


FIG. 15.—Relation between Crushing Strength per square inch and Ratio of Cross-section to Height of Specimen. (Bauschinger.)

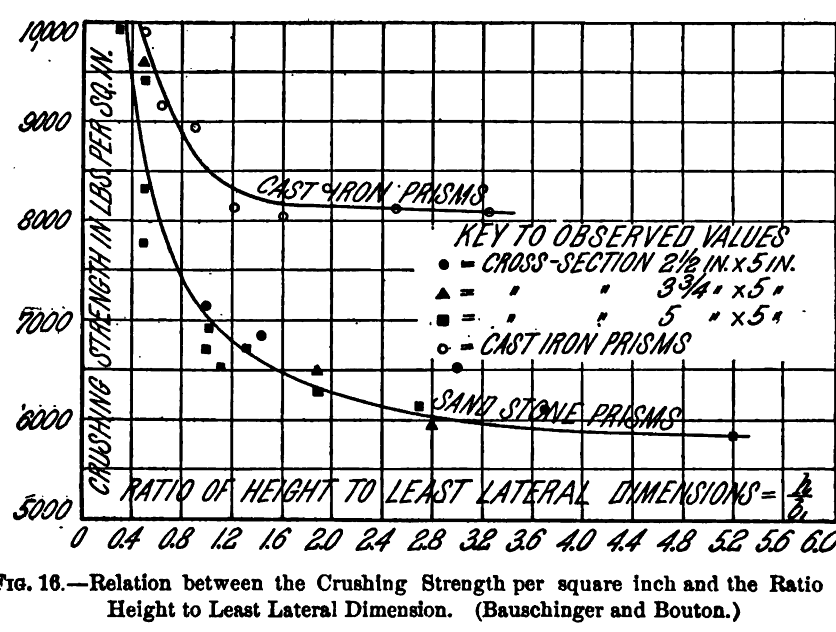


FIG. 16.—Relation between the Crushing Strength per square inch and the Ratio of Height to Least Lateral Dimension. (Bauschinger and Bouton.)

evident that formula (8) fits the results very well, the equation of the full line being

$$p = 5600 + 1400\frac{\sqrt{A}}{h} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

in pounds per square inch.

If a simpler formula is desired, the following may be chosen:

$$p = k + k'\frac{b_1}{h}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

where b_1 = least lateral dimension.

Fig. 16 shows how well this law fits the observations, the equation for this locus being, for the tests on sandstone,

$$5500 + 1565\frac{b_1}{h}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

The lower curve in Fig. 16 represents the law for sandstone prisms, and the upper one the law for cast-iron cylinders, when the strength argument on the diagram is multiplied by ten. The experiments for the former were made by Prof. Bauschinger, for the latter by Mr. Bouton. Mr. Bouton made his tests on two kinds of cast iron, using five bars of each and turning from these ten bars nearly one hundred cylinders. The tests on the longer cylinders have been excluded from the results plotted, as their length caused them to bend greatly, and hence their failure did not follow the law for short prisms. The plotted points on the tests of cast iron represent the average results of the number of similarly proportioned cylinders. In these tests there seems to be a possible minimum point at about $\frac{h}{d} = 1.5$, this being about the height which equals tan θ, or the least height offering an opportunity for failure on the theoretical angle. Why this should be the case does not appear, and the mean curve has been drawn without showing such a minimum indication.

22. Relative Strength of Prisms and Cubes.—In order to show the relation of the strength of a prism to that of a cube Bauschinger's observations were used, as plotted in Fig. 16 to p and $\frac{h}{b_1}$, and the curve as shown in Fig. 17 is the result.*

Thus, from this mean curve, we have the equation

$$\frac{\text{strength of prism}}{\text{strength of cube}} = 0.778 + 0.222\frac{b_1}{h}, \quad . \quad . \quad . \quad . \quad (12)$$

where b_1 = least lateral dimension, and h = height of prism.

This equation shows that the strength of a stone prism whose height is

* This law holds between the limits $h = 0.4b$ and $h = 5b$, these being the limits of the observations.

one and one half times its least lateral dimension has a strength equal to 92% of the strength of a cube of the same material.

This height of $\frac{h}{b_1} = \frac{3}{2}$ was found to be necessary to allow the material to shear on the theoretical angle of $45° + \frac{\phi}{2}$. Hence when the cubical form is used for test specimens in crushing, the results are 9% greater than if the proper height of specimen had been chosen.

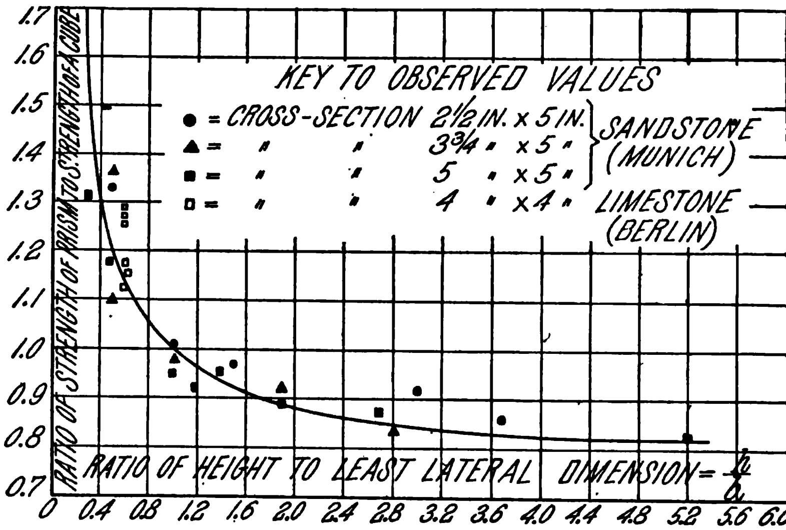


FIG. 17.—Relation between the Crushing Strength of Prisms and Cubes.

Also, if a brick, for instance, be tested flatwise, in which position $\frac{b_1}{h} = 2$, we find from this curve it will give a result 22% greater than that for a cube, and 33% greater than that for a specimen in which $\frac{b_1}{h} = \frac{2}{3}$. In other words, the results from tests on cubes are 9% too large, and on bricks flatwise they are 33% too large.

It will also be noted that, so far as these tests go, *the unit strength of the material is no function of the size of the specimen, but only a function of its form.*

23. Effects of Loading a Portion of the Cross-section.*—(*d*) *Chamfered Edges.*—If the edges of a cube or prism be chamfered off as shown in Fig. 18, and the load applied uniformly over the reduced area, the law of

* All the tests discussed in this article are taken from Prof. Bauschinger's published reports, but the author of this work has discussed them with the results as given.

the variation of strength with varying areas of compressed surface is shown by the curves on this figure.

Thus, as the area of pressed surface approaches that of the full cross-section, the load carried per unit of pressed surface decreases, as shown by the curved locus at the top, while the average load on the full cross-section

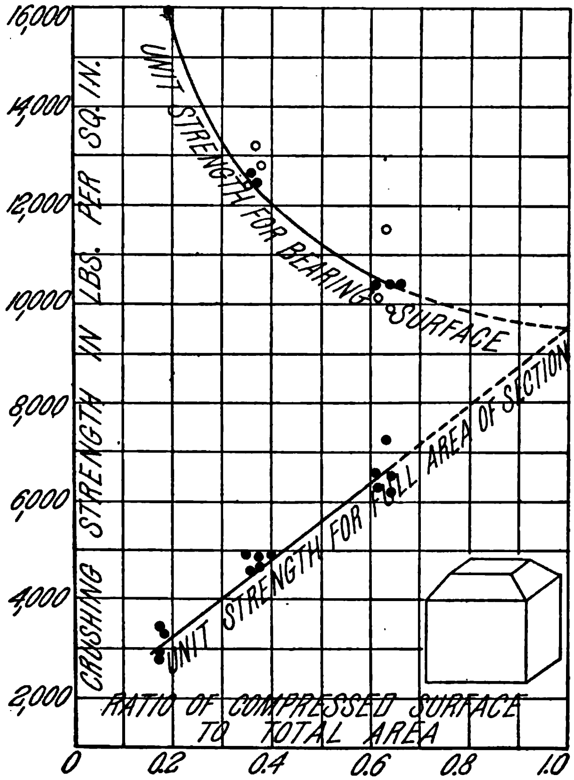


FIG. 18.—Crushing Strength of Cubes with Chamfered Edges. (Bauschinger.)

increases uniformly, as shown by the straight locus of Fig. 18, the two loci meeting at 9500 pounds, the strength per square inch of a full cube.

These results show clearly that the bearing surface should be that of the full cross-section of the specimen if normal results are to be obtained. The contrary has sometimes been asserted—that the strength of the specimen was not increased appreciably by the material outside the bearing surface. In other words, *crushing-test specimens should be true prisms in form, without chamfered edges or rounded corners.*

Since the locus of unit strength for bearing surface, Fig. 18, comes nearly into a horizontal direction as the pressed surface approaches the full area of cross-section, it follows that when the pressed surface is nearly equal to that of the full cross-section of the specimen the error introduced

by considering only the pressed surface is very small. For instance, if the area of the compressed surface is 0.8 that of the full cross-section (dimensions of cross-section 0.9 those of the full section), the error introduced by considering the pressed surface only would be by this curve $\frac{32}{1000} = 3.2\%$.

(*b*) *Square Bearing, Symmetrically Placed.*—When the pressed surface is square and placed symmetrically on a larger cube, the relation of the resistance per unit of pressed surface to the strength of the cube is shown on Fig. 19. Here the curves are given for the small bearing on one side

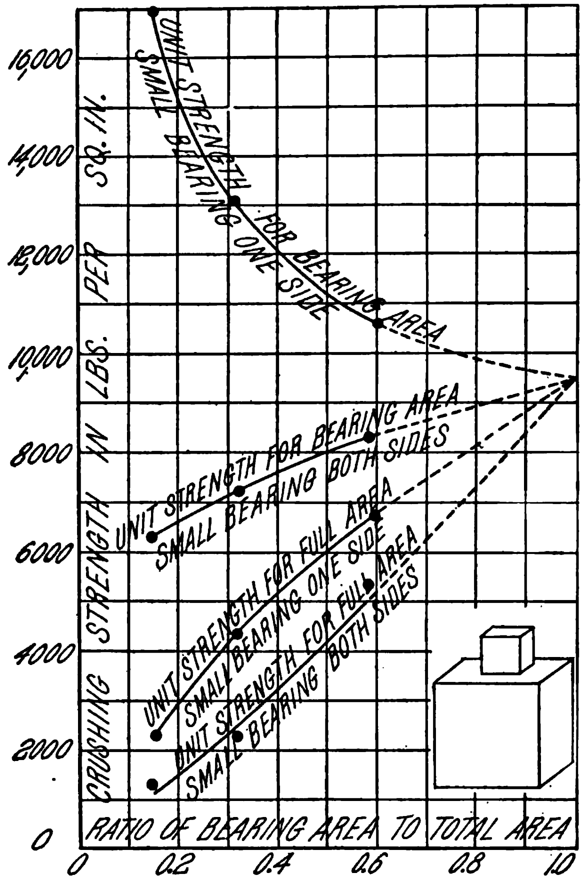


FIG. 19.—Effect of Loading a Portion only of the Surface of a Cube. (Bauschinger.)

and also on opposite sides, and the crushing resistance computed and plotted per unit of bearing surface and also per unit of cross-section of the cube. Evidently the loci must all meet at a point where the bearing area equals the total area on each side, and this point will be the strength of a cube of this material, which was 9500 pounds per square inch, the same as shown in Fig. 18, the material being the same.

(*c*) ***Bearing Surface Rectangular and Extending Entirely Across the Cube.***—In this case the resistance per square inch is a function of the distance of the pressed surface from the edge of the cube. This law is shown in Fig. 20. The material being the same as before, the strength of a cube

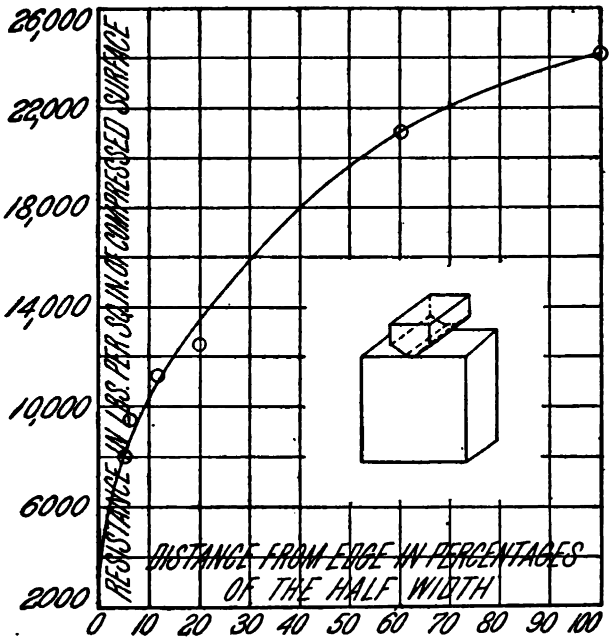


FIG. 20.—Effect of Loading a Zone on the Surface of a Cube. (Bauschinger.)

would be 9500 pounds per square inch. This corresponds to a distance from the edge of the cube equal to 8% of the half-width. As the bearing surface had a width equal to 10% of the half-width of the specimen, it follows that the outer line of the pressed surface came within 3% of the half-width, or 1½% of the total width from the edge of the specimen when the normal strength of the material was developed.*

24. General Laws of Crushing Strength.—The laws of crushing strength shown in Figs. 15 to 20 apply specifically to a particular quality of sandstone. In Fig. 16 it is shown that cast iron follows a different law. In all probability each kind of material, or at least materials which have different angles of rupture (that is to say, different coefficients of friction), will show different curves for the several relations indicated in these plates. In the absence of any more definite information, however, on this subject, it is thought the curves shown upon these plates will serve to indicate in a general way the laws of the variation of crushing strength with the varying conditions here indicated.

* See figures 12 and 18 for methods of failure for cases (*a*), (*b*), and (*c*).

By referring to Fig. 14 it will be observed that the cylinders all swelled more or less in the middle before rupture occurred. This is doubtless due to the restraining action of the friction against lateral motion on the end bearing surfaces. It is difficult to take this source of strength fully into account in a theoretical analysis of resistance to crushing.

25. Strength of Columns.—When a compression member is so long as to fail in compression by lateral deflection, its failure is a function of the elastic-limit strength and of the stiffness (modulus of elasticity) of the material, rather than of the ultimate strength of the material in compression. The discussion of this case properly comes in works on mechanics and on framed structures. The author has fully expressed his views on this question in his work on *Modern Framed Structures*, and to some extent in Chapter XVI of this work, and hence he will not occupy space with it here.

26. Weakening Effects of Eccentric Loading.—Few persons are aware of the great increase of stress on the near side of a member subjected to a direct stress (either tension or compression) caused by an eccentricity of the load-line with reference to the gravity-axis of the member. This eccentricity may result from an eccentric imposition of the load itself; or from the member being bent; or from the addition of material on one side of the member, such addition usually proving a source of weakness instead of strength. These three cases are shown in Fig. 21. In each case we have

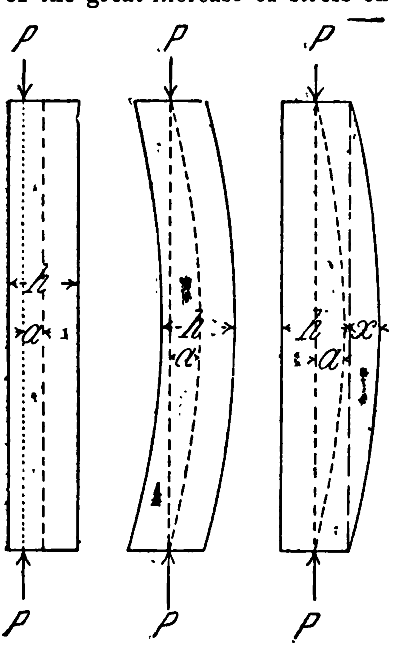


Fig. 21.

Total load $= P$;
" area $= A$;
Eccentricity $= a$;
Width $= h$;
Moment of inertia of section $= I$;
Radius of gyration of section $= r$;
Distance of extreme fibre from the gravity-axis $= y_1 = \frac{h}{2}$ with symmetrical sections;

Total stress on nearest outer fibre $= f$.

Hence we have for symmetrical cross-sections, where $y_1 = \frac{h}{2}$,

$$f = \frac{P}{A} + \frac{P}{A}\left(\frac{ah}{2r^2}\right) = \frac{P}{A}\left(1 + \frac{ah}{2r^2}\right).^* \quad . \quad . \quad . \quad . \quad . \quad (13)$$

* The stress due to the bending moment Pa is found from the equation $m = \frac{fI}{y_1}$ or $f = \frac{my_1}{I}$, where $m = Pa$, and $I = Ar^2$.

For solid rectangular cross-sections we have $r^2 = \frac{I}{A} = \frac{h^2}{12}$; hence for such sections

$$f = \frac{P}{A}\left(1 + \frac{6a}{h}\right). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

The proportionate increase in the stress, therefore, over that which would obtain for a concentric load is given by the fraction $\frac{6a}{h}$. In other words, when $a = \frac{1}{6}h$ the stress on the outer fibre on the near side is doubled, compared with that for a central loading.

To discover the weakening effect of additional material added to one side of a member, assume a central loading on a straight symmetrical member having an initial width $= h$, $(a = o)$. If additional material, to a thickness of x, be now added on one side of this member, the new total width becomes $h + x$, and the eccentricity is $a = \frac{x}{2}$. Assuming the member to be solid rectangular in cross-section, with original dimensions of b and h, the new dimensions of section are b and $h + x$; the former area was $A = bh$, and the latter $A' = b(h + x)$. Before the addition we should have $f = \frac{P}{A} = \frac{P}{bh}$. After the addition we should have, from (14), for the stress on the near side,

$$f' = \frac{P}{A'}\left(1 + \frac{6a}{h'}\right) = \frac{P}{b(h + x)}\left(1 + \frac{3x}{h + x}\right). \quad . \quad . \quad . \quad . \quad (15)$$

Hence the increase of the stress due to an unsymmetrical addition of material is

$$f_t = f' - f = \frac{P}{b}\left(\frac{1}{h + x} + \frac{3x}{(h + x)^2} - \frac{1}{h}\right). \quad . \quad . \quad . \quad . \quad (16)$$

This is zero for $x = 0$ and for $x = 2h$, and it is a maximum for $x = \frac{h}{2}$, when it becomes

$$\max. f_t \left(\text{for } x = \frac{h}{2}\right) = \frac{1}{3}\frac{P}{bh} = \tfrac{1}{3}f. \quad . \quad . \quad . \quad . \quad . \quad (17)$$

Hence we may say that *the addition of material on one side of a member subjected to a direct stress symmetrically placed weakens it until the added material has exceeded twice the original thickness of the member, the maximum weakening occurring when the added material is one half the original thickness, when the enlarged member is only three fourths as strong as the original member.**

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* Attention was called to this fact by Mr. Carl G. Barth in *Trans. Engrs. Club of Phila.*, Oct. 1891, p. 307.

CHAPTER IV.

MATERIALS UNDER SHEARING STRESS.

27. Two Manifestations of Shearing Stress.—When all the opposing external forces which act on a body lie in one plane,* but not in one and the same line, the resisting stresses are those of simple shear and cross-bending, without torsional stress.

When the opposing external forces do not lie in one plane the resisting stresses are those of torsional shear, with or without cross-bending and simple shear.

In any case these three kinds of stress are determined separately, as follows:

(*a*) *For Parallel External Forces in One Plane.*—The moment of resistance of the bending (direct) stresses at any transverse section is equal to the algebraic sum of the moments of the external forces on either side of that section taken about the neutral axis in that section.

The simple shearing stress on any section is equal to the algebraic sum of the transverse components of the external forces on either side of that section.

(*b*) *For Parallel External Forces Not in One Plane.*—First replace all the forces by equal parallel forces acting in the plane of the axis of the body, and by couples equal in value in each case to the force multiplied by its displacement. Then the moments of resistance and the simple shearing stresses will be the same as in the last case, and in addition there will be the moment of torsion.

The torsional moment at any transverse section is equal to the algebraic sum of the moments of the couples of the displaced forces, acting on either side of the transverse section in question.

(*c*) *For Non-parallel Forces Acting in Any Manner.*—Resolve all forces into horizontal and vertical components at their points of application, and then solve for bending moments, shears, and torsions at any section in these two planes.

The bending moment at this section will then be the square root of the sum of the squares of the bending moments at right angles to each other.

* When a force is distributed over an area it is here supposed to act at the centre of gravity of these force-elements.

The total shear will also be the square root of the sum of the squares of the primary shears at right angles to each other.

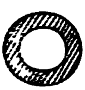
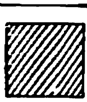
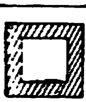
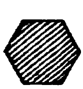
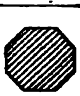
The total moment of torsion will be the algebraic sum of the two moments of torsion found from the two sets of forces.

28. The Moment of Torsion gives rise to a shearing stress over the entire cross-section, which is zero at the centre of gravity of the section, and which increases in intensity directly as the radial distance from the gravity axis.

For various forms of sections, the following intensities of shearing stress are found, by the principles of mechanics, for the corresponding forms of cross-section.

The general equation for resistance to torsion is

$$M = \frac{sJ}{r}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Figure.	Dimensions.	Area.	J.*	r	$\frac{J}{r}$
	Radius $= r$	πr^2	$\frac{\pi r^4}{2}$	r	$\frac{\pi r^3}{2}$
	Outer radius $= r$, Inner " $= r_1$	$\pi(r^2 - r_1^2)$	$\frac{\pi(r^4 - r_1^4)}{2}$	r	$\frac{\pi(r^4 - r_1^4)}{2r}$
	Side $= b$	b^2	$\frac{b^4}{6}$	$\frac{b}{2}\sqrt{2}$	$\frac{b^3}{3\sqrt{2}}$
	Outer dimension $= b$, Inner " $= b_1$ ...	$b^2 - b_1^2$	$\frac{b^4 - b_1^4}{6}$	$\frac{b}{2}\sqrt{2}$	$\frac{b^4 - b_1^4}{3\sqrt{2}}$
	Side $= a$.	$\frac{3a^2}{2}\sqrt{3}$	$\frac{5a^4}{8}\cdot\sqrt{3}$	a	$1.082a^3$
	Radius of circumscribed circle $= r$	$2r^2\sqrt{2}$	$r^4(1 + 2\sqrt{2})$	r	$1.276r^3$
	Longer axis $= 2a$, Shorter axis $= 2b$	πab	$\frac{\pi}{4}(a^2b + b^2a)$	a	$\frac{\pi b}{4}(a^2 + b^2)$
	Longer semi-axes $= a$ & a_1, Shorter " " $= b$ & b_1	$\pi(ab - a_1b_1)$	$\frac{\pi}{4}\left[\begin{matrix} ab(a^2+b^2) \\ -a_1b_1(a_1^2+b_1^2) \end{matrix}\right]$	a	$\frac{\pi}{4}\left[\begin{matrix} b(a^2+b^2) \\ -\frac{a_1b_1}{a}(a_1^2+b_1^2) \end{matrix}\right]$

where M = total torsional moment;

s = shearing stress on extreme fibre;

J* = polar moment of inertia of cross-section about the gravity axis;

r = distance from neutral axis to the extreme fibre having the shearing stress s.

Whence we have, for the forms figured, the relations given in the table.

29. Shearing Deformations.—As shown in Arts. (7) and (8), a shearing action of external forces results in angular deformation of the body. In the case of simple shear, or where the forces lie in one plane, the angular deformation from shear is very small, the bending being mostly due to the longitudinal deformations resulting in the direct tensile and compressive resisting stresses on the two sides of the neutral plane respectively. When the forces do not lie in one plane, or where there is a moment of torsion, the angular deformation gives rise to a twist of the body about the neutral longitudinal axis. Thus in Fig. 23 assume the solid cylinder, anchored at o, to have a length l and a radius r. Let the torsional moment be $Pa = M_t$. Then the shearing stress on the extreme fibre is, by equation (1),

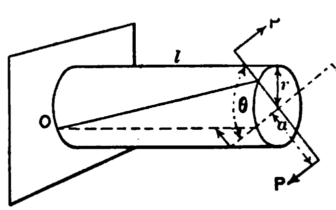


FIG. 23.

$$s = \frac{M_t r}{J} = \frac{2Pa}{\pi r^3}, \quad \ldots \ldots \ldots \ldots \quad (2)$$

where J is the polar moment of inertia = twice the rectangular moment of inertia in this case.

In Art. 9 the shearing modulus of elasticity was defined as

$$E_s = \frac{\text{shearing stress per sq. in.}}{\text{angular deformation}}.$$

If we take the stress and angular deformation of the outer fibre in Fig. 23, we have:

$$\text{Shearing stress per sq. in.} = s = \frac{2Pa}{\pi r^3}.$$

$$\text{Tangent of the deformation angle} = \frac{r\theta}{l} = \text{deformation angle,}$$

since this angle is small.

Hence we have

$$E_s = \frac{2Pal}{\pi r^4 \theta} = \frac{sl}{r\theta}, \quad \ldots \ldots \ldots \ldots \quad (3)$$

* The student's attention is called to the fact that the polar moment of inertia is equal to the sum of the true rectangular moments of inertia about the principal axes through the centre of gravity of the section.

or

$$\theta = \frac{2Pal}{\pi r^4 E_s} = \frac{sl}{rE_s}. \quad \text{. . . . . (4)}$$

In general, for any cross-section we have

$$E_s = \frac{M_t l}{J\theta} = \frac{sl}{\theta y_1}, \quad \text{or} \quad \theta = \frac{M_t l}{E_s J} = \frac{sl}{E_s y_1}, \quad \text{. . . . . . (5)}$$

where y_1 = distance from the neutral axis to the extreme fibre in which the stress is s.

In Art. 9 it was shown that the shearing modulus of elasticity = $\frac{2}{5}$ of Young's modulus, or $E_s = \frac{2}{5}E$. Hence in terms of Young's modulus of elasticity, which is that ordinarily given, we have

$$\theta = \frac{5}{2} \cdot \frac{M_t l}{EJ} = \frac{5sl}{2Ey_1}, \quad \text{. . . . . (6)}$$

where θ = angular movement in terms of the radius;
M_t = torsional moment on the bar;
l = length of bar between sections representing a relative angular movement of θ;
s = shearing stress on outer fibre;
E_s = shearing modulus of elasticity of the material;
E = the ordinary modulus of elasticity;
J = polar moment of inertia = $I_x + I_y$, where these are the rectangular moments of inertia about the principal axis through the centre of gravity of the section;
y_1 = distance from neutral axis of outer fibre in which the shearing stress is s.

CHAPTER V.

MATERIALS UNDER CROSS-BENDING STRESS.

30. Historical Sketch.*—For two hundred and fifty years the true theory of the strength of a beam has been a much-mooted question amongst physicists, engineers, and mathematicians.

Galileo was the first of whom we have any record who undertook to discuss the problem. In his famous Dialogues (Leiden, 1638, from which Fig. 24 is taken) he

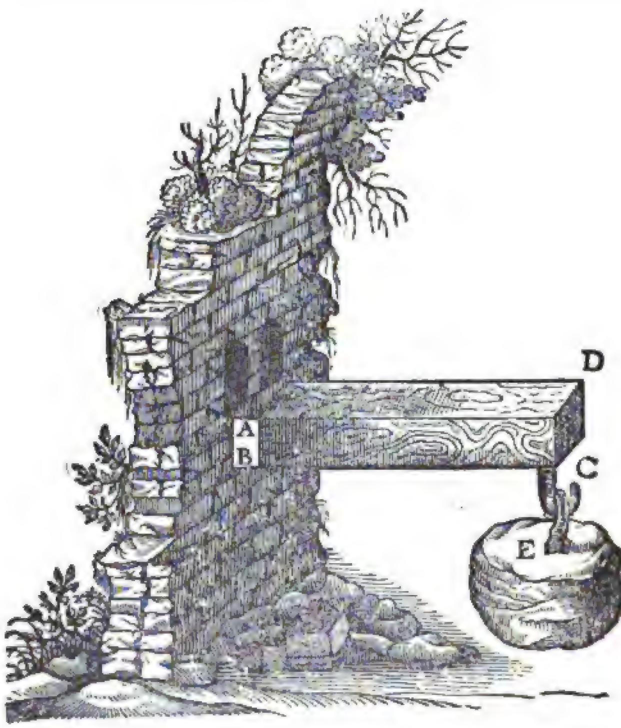


FIG. 24.

propounds a theory based on an assumed absolute rigidity of the material, and concluded that the fibres of the beam were subjected to a uniform tension which acted about the base of the beam as a fulcrum. On this theory the moment of resistance

* This historical review of the development of the true theory of the beam is derived mostly from Saint-Venant's *Historique Abrégé des Recherches sur la Résistance et sur l'Élasticité des Corps Solides*, prefixed to his Navier's "Leçons," Third Edition, Paris, 1864, and from Todhunter's *History of the Theory of Elasticity*, Cambridge, Eng., 1886. It is here reprinted from the author's joint work on *Modern Framed Structures*.

of a solid rectangular beam would be $\frac{fbh^2}{2}$, where f is the ultimate strength of the material in tension.

Robert Hooke first published his famous law of the relation between deformation and stress in 1678, discovered by him he says 18 years previously, and kept secret for the purpose of procuring patents on some applications of the principle to springs for watches, clocks, etc. Two years previously he had ventured to publish the law in an anagram at the end of another book, in this form, "*c i i i i n o s s s t t u v*," which being interpreted reads, "*Ut tensio sic vis*," or, "as the extension so is the resistance." Hooke makes this law apply to all "springy" bodies, amongst which he names nearly all ordinary solids. This is still known as *Hooke's Law*.

Mariotte showed by experiment in 1680 that the fibres on one side of the beam were extended and on the other side compressed, and *assumed* that the neutral surface passes through the centre of gravity of the section.

Varignon, in 1702, undertakes to harmonize the theories of Galileo and Mariotte, by admitting the extension of the fibres, but puts the neutral plane at the bottom, as Galileo did, and assumes the tensile stress as uniformly varying from there to the other side. This would make the strength of a solid rectangular beam $\frac{fbh^2}{3}$, which agrees almost exactly with the facts for cast iron at rupture when f is the tensile strength.

James Bernouilli made an important advance by applying Mariotte's law to obtain deflections of beams (1694 and 1705), and argued that the position of the neutral axis is a matter of indifference, which was a great error. He denied the truth of Hooke's law, which we know is not applicable to all substances, nor to the point of rupture with any substance. He first constructed stress diagrams, but his work in the field of hydraulics was of even greater importance than in the study of solids.

A. Parent, a French academician, seems to have been the first to perceive (1713) the mechanical necessity of equilibrium between the tensile and compressive stresses, which condition, together with that of a uniform variation of stress, fixes the position of the neutral axis at the centre of gravity of the section. This important discovery seems, however, to have passed unnoticed.

Coulomb reannounced this relation in a memoir to the French Academy in 1773, or sixty years after its first publication by Parent. Saint-Venant credits Coulomb with never having seen Parent's work, as no writer of that century has mentioned it. But even after this second publication of so important a necessary truth, such workers as Girard, Barlow, and Tredgold all misconceived the mathematical necessities in the problem, and resorted to various makeshifts to explain the strength of beams.

Navier finally, in 1824, put the matter on a solid mathematical basis, although he also at first went entirely astray. He stated in his first edition that the moment of resistance varied as the cube of the depth of the beam, and in his second edition this error was corrected, but the moment of the stresses on one side of the neutral axis was said to be equal to the moment of the stresses on the other side, about that axis, an equality which does not exist except on symmetrical sections. Navier also fully developed the theory of the deflection of beams as we now use it.

Saint-Venant, a student of Navier's, has finally (1857) in his notes on Navier's *Leçons* given a complete analysis of both the elastic and the ultimate strength of a beam, with suitable equations which will give theoretical results agreeing with the actual tests, when the empirical constants are properly evaluated. This great engineer, physicist, and teacher has done more than any other one to bring theory and practice into harmony and to put both on a thoroughly scientific basis, so far as the strength and elasticity of engineering materials is concerned.*

In spite of these various true expositions of this subject the source of strength in a beam continues still to be very imperfectly understood by

* He died January 6, 1886.

many engineers, and even by current writers on applied mechanics, and gross errors in this direction are still common. It is in consideration of this state of the science that the problem is treated so fully here.

31. Fundamental Equations of Equilibrium.—When a solid body is in equilibrium under the action of non-concurrent external forces, the following propositions hold true for the body as a whole:

I. *The sum of the vertical components of the external forces is equal to zero.*

II. *The sum of the horizontal components of the external forces taken in any plane is equal to zero.*

III. *The sum of the moments of the external forces taken about any point is equal to zero.*

When a solid body is subjected to the action of non-concurrent forces acting in one plane the body may be regarded as a beam, since the effect of the external forces is to bend the body and develop in it what are commonly called cross-bending stresses. If a section be passed through the body perpendicular to the plane of the forces, and the portion of the body on one side of this section be removed, the other portion may be held in equilibrium with the external forces acting upon it, by means of the stresses existing in the body on this cross-section, these stresses now being regarded as external forces, as indicated in Fig. 25. Since the remaining portion of the body now under consideration is in equilibrium under the action of external forces and of internal stresses, which for the time may be regarded as external forces, the three propositions given above will apply. Or, stating these propositions now so as to equate the real external forces with the internal stresses developed at the section, they would read as follows:

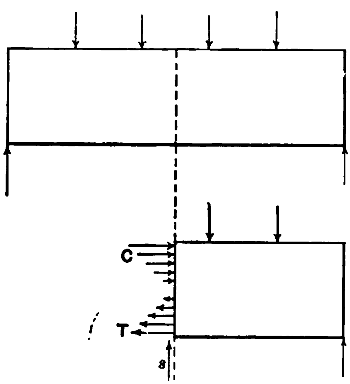


FIG. 25.

If a transverse section be passed through a beam—

I. *The sum of the vertical components of the stresses acting at the section is equal to the sum of the vertical components of the external forces acting upon the body on either side of that section.*

II. *The sum of the horizontal components of the stresses acting on the section is equal to the sum of the horizontal components of the external forces acting upon the body on either side of that section.*

III. *The sum of the moments of the stresses acting on that section is equal to the sum of the moments of the external forces acting on the body on either side of that section.*

It follows from the above that if all the external forces acting upon

a beam are parallel vertical forces, the end reactions or supports being regarded as external forces the same as any primary weights or loads, and if no horizontal forces act upon the beam, then we should have for any vertical section—

I. *The shearing stress is equal to the algebraic sum of the external forces acting on either side of the section.*

II. *The algebraic sum of the horizontal stresses acting on the section is equal to zero.*

III. *The algebraic sum of the moments of the stresses acting on that section, which is commonly called the moment of resistance, is equal to the sum of the moments of the external forces about any point in that section.*

The effect of the action of cross-bending forces upon a beam is to bend or deflect it, thus shortening the lengths of the fibres or elements on the concave side of the beam, and lengthening them on the convex side. So long as this action does not exceed the elastic limits of the material, the resisting stresses are directly proportional to the deformations. Hence there is always found a compressive stress on the concave side and a tensile stress on the convex side of a beam, and therefore there will be a plane near the centre of the beam the elements of which are neither lengthened nor shortened, and on which there will be no longitudinal stress. This is called the *neutral plane* or "*neutral axis*" of the beam.

Furthermore, a *geometrical* effect of the bending of a beam is to produce deformations which are zero at the neutral plane and which increase uniformly outward to the extreme convex and concave sides, and hence the longitudinal resisting stresses developed by these deformations also increase uniformly outward. Within the elastic limits, therefore, *the direct stresses increase uniformly from the neutral plane to the extreme fibres.*

Since from Proposition II, as stated above, the summation of the horizontal stresses on the cross-section is zero, in simple cross-bending, where the external forces have no horizontal components, it follows that *the total summation of the tensile stresses on the convex side of the neutral plane must always exactly equal the total summation of the compressive stresses on the concave side.* Also by Proposition III the sum of the moments of all these stresses taken about any point in this plane must equal the sum of the moments of the external forces acting on either side of the section taken about the same point. If this centre of moments be taken in the neutral plane itself it will at once be evident that the moment of the tensile forces on one side has the same sign as the moment of the compressive forces on the other side, and that they are, therefore, to be added together numerically in order to equal the algebraic sum of the moments of the external forces acting on either side of the section. While, therefore, the sum of the moments of the tensile stresses *may* be numerically equal to the sum of the moments of the compressive stresses (which is the case for symmetrical cross-sections), yet since they are to be added together numerically, in order

to equal or hold in equilibrium the moments of the external forces on one side of the section, there is evidently no mathematical necessity why the moments of the compressive stresses should equal the moments of the tensile stresses; and in unsymmetrical sections, and even in symmetrical sections beyond the elastic limit, these moments are not equal to each other.

Since the stresses on any cross-section of a beam subjected to the action of bending forces increase uniformly from the neutral plane to the extreme sides, it is evident that it is only the stress found to exist in the extreme fibres or elements of the beam which needs to be determined. That is to say, if the maximum stresses are kept within the working limits, it is immaterial what the particular stresses are on other portions of the cross-section. It is common, therefore, to find the relation between the total moment of resistance of a beam (which of necessity is always numerically equal to the bending moment of the external forces), and the stresses on the extreme fibres or elements of the cross-section of the beam. This general relation between the bending moment and the stresses on the extreme fibres is made the subject of the following article.

32. Relation between the Moment of Resistance and the Stress on the Extreme Fibre.—In Fig. 26 let the load P be applied at C, and this will

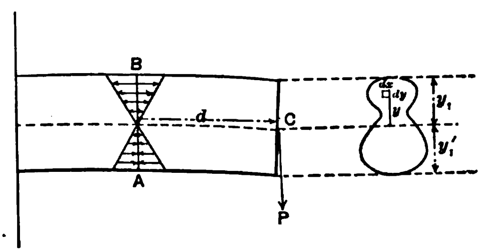


FIG. 26.

produce a bending moment on AB of Pd. On this plane the moment of the longitudinal stresses makes up the moment of resistance which holds in equilibrium (and hence is always numerically equal to) the bending moment of the external forces. That is to say, $M = Pd = M_0$, the moment of resistance. We shall here assume the cross-section to be irregular and unsymmetrical, as shown in the figure. The direct stress varies uniformly across the section in all cases. The following notation will be used :

M = bending moment of the external forces.

M_0 = moment of resistance of the direct stresses = M.

p = intensity of the direct stress at the distance y from the neutral plane = ay, where a = intensity of direct stress at a unit's distance.

f = intensity of the direct stress at the extreme side of the beam.

y_1 = distance of extreme fibre on one side from the neutral axis.

y_1' = " " " " " the other side from the neutral axis.

$I = y^2dxdy$ = moment of inertia of the cross-section about the centre of gravity axis.

$\bar{y}$ = distance from axis of reference to the centre of gravity of the cross-section.

Intensity of stress on *any* fibre $= p = ay$; . . . . (1)

Total stress on fibre having an area of $dxdy = pdxdy = aydxdy$; . . . (2)

Moment of stress on fibre $dxdy = pydxdy = ay^2dxdy$; . . . . . . (3)

Total moment of all stresses $= M_0 = \Sigma ay^2dxdy = a\int_{-y_1'}^{+y_1} y^2dxdy = aI$. . (4)

But as $p = ay$, so $f = ay_1$ and $f' = ay_1'$; or

$$a = \frac{f}{y_1} = \frac{f'}{y_1'}.$$

Therfeore

$$M = M_0 = aI = \frac{fI}{y_1}^* = \frac{f'I}{y_1'}. \quad . \; . \; . \; . \; . \; . \; . \quad (5)$$

This is the general equation between the moment of resistance and the stress on either extreme fibre. When the section is symmetrical, $y_1 = y_1'$; hence $f = f'$, and only one side need be considered.

When the cross-section is solid and rectangular, equation (5) becomes

$$M_0 = \tfrac{1}{6}fbh^2.^* \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (6)$$

The above demonstration assumed that the neutral axis or plane of the beam passed through the centre of gravity of the cross-section, since I was referred to this gravity axis. This remains to be proved.

From equation (2) we have, the stress on any element is $aydxdy$, where y is measured from the neutral axis. But for simple cross-bending the algebraic sum of these direct stresses over the whole section is zero; hence we have

$$\int_{-y_1'}^{+y_1} aydxdy = a\int_{-y_1'}^{+y_1} ydxdy = a\int_{-y_1'}^{+y_1} ydA = 0. \quad . \; . \; . \; . \quad (7)$$

But

$$\int ydA = \bar{y}A,\dagger \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (8)$$

---

* Both equation (5), $M_0 = \frac{fI}{y_1}$, for any section, and equation (6), $M_0 = \frac{1}{6}fbh^2$, for solid rectangular section, should be *thoroughly memorized* by the student, as they are of constant application.

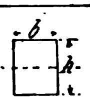
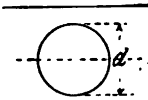
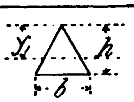
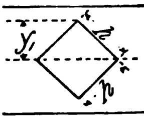
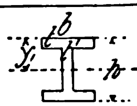
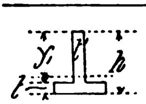
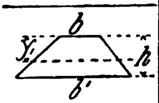
† The symbol $\bar{y}$ denotes the distance from the axis of y to the centre of gravity of the cross-section, and it equals $\frac{\int ydxdy}{\int dxdy} = \frac{\int ydxdy}{A}$.

since the sum of the statical moments of the elementary areas about any axis is equal to the moment of the total area into the distance to its centre of gravity. Therefore we have, for reference to the neutral axis,

$$a\int_{-y_1'}^{+y_1} y\,dA = 0, \quad \text{or} \quad \bar{y}A = 0. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (9)$$

But yA can only equal zero when reference is made to the gravity axis. Therefore these two axes must coincide. In other words, *the neutral plane always traverses the centre of gravity axis of the beam, so long as the stresses remain inside the elastic limits of the material in both tension and compression, and also provided the modulus of elasticity is the same for both kinds of stress.*

33. Moments of Resistance (Strength) of Beams of Various Forms of Cross-section.—The moment of resistance of a beam of any form of cross-

Form of Cross-section.	Distance of Centre of Gravity, or Neutral Axis, from the Most Distant Fibre. $= y_1$	Moment of Inertia about the Centre of Gravity of the Section. $= I$	Moment of Resistance in Terms of the Stress in the Most Distant Fibre. $= M_0 = \frac{fI}{y_1}$
	$\frac{h}{2}$	$\frac{bh^3}{12}$	$\frac{1}{6}fbh^2$
	$\frac{d}{2}$	$\frac{\pi d^4}{64}$	$\frac{\pi}{32}fd^3$
	$\frac{2}{3}h$	$\frac{bh^3}{36}$	$\frac{1}{24}fbh^2$
	$\frac{h}{2}\sqrt{2}$	$\frac{h^4}{12}$	$\frac{1}{6\sqrt{2}}fh^3$
	$\frac{h}{2}$	$\frac{bh^3 - (b - t')(h - 2t)^3}{12}$	$\frac{bh^3 - (b - t')(h - 2t)^3 f}{6h}$
	$\frac{\frac{1}{2}t'h^2 + t(b - t')(h - \frac{1}{2}t)}{t'h + t(b - t')}$	$\frac{bh^3 - (b - t')(h - t)^3}{3} - Ay_1^2$	$\frac{fI}{y_1}$
	$\frac{b + 2b'}{b + b'} \cdot \frac{h}{3}$	$h^3\left[\frac{3b + b'}{12} - \frac{(b + 2b')^2}{18(b + b')}\right]$	$\frac{fh^2}{6}\left[\frac{8(3b + b')(b + b')}{2(b + 2b')} - (b + 2b')\right]$

section was found to be, by equation (5), $M_0 = \frac{fI}{y_1}$, where f = intensity of stress on the extreme fibre which lies at a distance from the neutral plane equal to y_1, and I is the rectangular moment of inertia of the cross-section about the neutral or gravity axis. In the table on p. 48 are given the values of y_1, I, and M_0 for various forms of sections which are commonly used as beams. For tabular and graphical methods of finding the moments of inertia of irregular forms, see *Modern Framed Structures*, pages 127–130.

The values given in the above table are true for all values of f inside the elastic limit. When this limit is exceeded the stress no longer varies uniformly across the section, but the stresses near the neutral axis are larger than the above theory allows, and hence, for a given actual stress on the extreme fibres beyond the elastic limit (as the breaking-stress, for instance), the moment of resistance is much more than would be obtained by using the breaking value of f (in tension or compression) and substituting this in the above formulæ. It must be understood, therefore, that *in no case are these formulæ true at rupture, but only inside the elastic limits of the material.* It is for this reason that the values of f as found from cross-bending tests carried to failure, and as computed from the above formulæ, differ so largely from the breaking values of the material in direct tension or compression.* Thus, cast iron, which has a tensile strength of 20,000 pounds per square inch and which breaks on the tension side in cross-breaking, has a value of f, when computed by the above formulæ from a breaking-load, of from 30,000 to 40,000 pounds per square inch, depending somewhat on the shape of the cross-section of the specimen. The more the material is concentrated near the neutral plane the more the value of f differs from the tensile strength. This value of f, computed from the breaking moment, is called the *modulus of rupture in cross-breaking.* It is from 1.5 to 2 times the tensile strength of the metal.

In timber beams the reverse is the case; that is to say, the crushing resistance being less than the tensile resistance, the modulus of rupture in cross-breaking is greater than the former and less than the latter, and it is in fact nearly a mean of the two.

34. Strength (Moment of Resistance) of Beams beyond their Elastic Limits.—After the stress on the extreme fibres on one or both sides of the beam has passed the elastic limit, the distribution of stress over the section is no longer uniformly varying as was assumed in deriving the formulæ of the last article, and the law of this variation will now be examined.

In all cases the variation of stress across the transverse section of a beam subjected to simple cross-bending, with or without shearing stress, follows the law of the variation of the stress ordinates to a stress-diagram

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* See a full discussion of this subject in the author's work on *Modern Framed Structures*, Chapter VIII.

in which the extreme ordinate represents the stress on the extreme fibre of the beam.

Thus in Fig. 28, suppose the beam to be cast iron, and to be bent until the stress on the extreme fibre on the tension side has become f_t. Passing

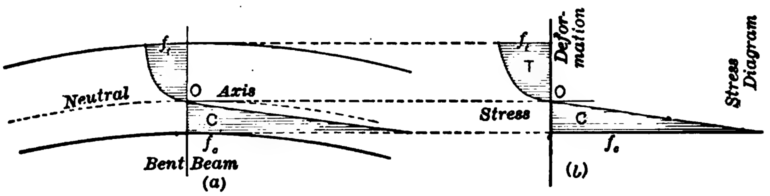

Fig. 28.

now to the tension portion of the stress-diagram for this material,* we see that this stress, f_t, is found far beyond the elastic limit of the metal in tension. Let us now recur to the fact that the *deformation* of the longitudinal fibres of the beam increases uniformly outward from the neutral axis, even beyond the elastic limit, since the section remains sensibly plane, and hence the uniform increase of the deformation is a geometrical necessity. In view of this fact it becomes evident that the law of increase of stress from the neutral axis outwards, or the law of the increments of stress corresponding to *equal* increments of deformation, is exactly that represented by the stress-diagram, since here we have the increments of stress shown for equal increments of deformation. Hence it follows that if f_t is the stress on the extreme fibre of the bent beam on the tension side the stresses on all other fibres on the tension side are truly indicated by the lengths of the corresponding ordinates on that side of the neutral axis, when the position of the stress ordinate f_t in the stress-diagram is taken as the position of the extreme tension side of the beam, and the origin in that diagram is taken as lying on the neutral axis of the beam. Evidently the same argument would apply to the compression side.

35. Distribution of Stress and Position of the Neutral Axis at Rupture.—In a brittle material like stone or cast iron, failure occurs on the tension side; while in the case of wood, failure usually occurs first on the compression side of the beam. The diagrams shown in Fig. 28 may fairly be taken as representing the facts in the case of cast iron, and those in Fig. 29 in the case of timber. Since timber is much stronger in tension than in compression, it fails first on the compression side. Furthermore, after the fibres have buckled, or broken down, in compression, they are able to support only about three fourths as much of a load as before, so that the compression stress-diagram has the peculiar form shown in the accompanying figure.

At failure, therefore, the tensile and compressive stresses are distributed over the section in a manner entirely different from that which obtains

* See Chap. XXIII for complete stress-diagrams for cast iron of various qualities.

within the elastic limit. The statement made in Art. 25, however, regarding the equality between the sums of the tensile and compressive stresses must still hold, as this is a mathematical or mechanical necessity; and as this total stress is graphically represented by the area of the stress-diagram

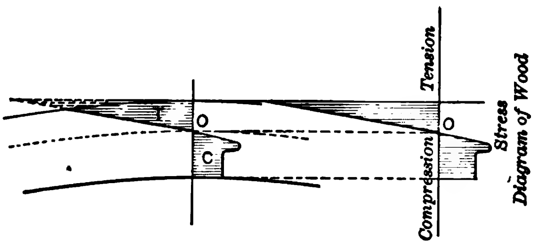

FIG. 29.

shown on the sections of the beams in Figs. 28 and 29, it follows that *these stress areas on the two sides of the neutral axis must be equal.*

Thus in the case of timber, for instance, the neutral plane at first lies in the centre of gravity of the cross-section, but after the material has begun to crush on the compression side, the neutral plane rapidly moves towards the tension side of the beam and often, at final rupture, it lies very near this side, the tension stress area being a triangle of very long base (stress on extreme fibre) and very short altitude (distance to neutral plane). It is evident that, although the beam has long since failed in compression, if it be continuously deflected, failure must ultimately occur also in tension. When the material is weaker in tension than in compression such double failure cannot occur, since the tension failure parts the body, and the rupture is complete. Evidently no general law can be given for distribution of the stress across the section after the elastic limit has been passed, other than to say it is that of the corresponding stress-diagrams of that material in direct tension and compression respectively.

36. Moduli of Rupture in Cross-breaking.—From the facts related in the preceding article it is evident that the formulæ of Articles 32 and 33 cannot apply at rupture, and that if the breaking-load be used for computing the so-called ultimate strength of the material in pounds per square inch (the "modulus of rupture in cross-breaking," and the quantity f in those formulæ when P is the breaking-load, or when M is the ultimate bending moment), the result obtained as the value of f is a purely fictitious quantity, and that it does not really represent any actual tensile or compressive stress on the extreme fibres at all. It may, however, be called the "modulus of rupture in cross-breaking" in pounds per square inch, and used to indicate the strength of the material when loaded as a beam; but it must not be confused with, or assumed to have any fixed relation to, either the tensile or the compressive strength of the material. As a matter of fact it always lies somewhere between these two latter values, but it does not have any uni-

versal relation to them. It is always dependent largely on the form of the cross-section of the beam, as to the concentration of material near the neutral axis or near the extreme sides. Thus the elastic-limit strength of a rolled I beam can be very closely approximated by using for f in equation (5) the tensile or compressive elastic-limit strength of the material in either tension or compression, while the elastic-limit strength of a solid round bar could not be determined very closely by so doing. Also the ultimate strength of a cast-iron beam of an I-shaped cross-section could be determined approximately by using the tensile strength of the material for the value of f on the tension side of the beam in eq. (5), but the ultimate strength of a round or square cast-iron bar would be nearly twice as much as would be shown by the use of eq. (5) if the tensile modulus of rupture were taken.

37. The Distribution of Shearing Stress in a Beam.—(*a*) *The Relation between Shear and Bending Moment at any Section.*—In Fig. 30 assume any two adjacent sections dx apart. Let the total shearing force acting here be S. Call the bending moment at the first section M, and that at the other M'. Assume the beam to be cut at the section where the moment is M, and the left portion removed and replaced by the direct tensile and compression stresses, and also by the total shear, S. Then it is evident the moment at the adjacent section is

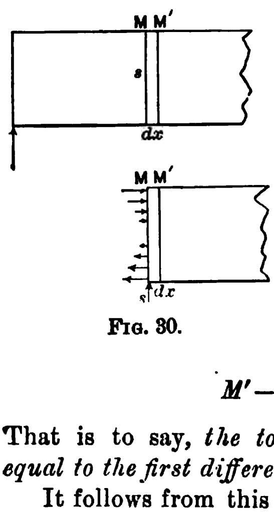

FIG. 30.

$$M' = M + Sdx. \quad . \quad . \quad . \quad . \qquad (10)$$

But

$$M' - M = dM,$$

hence we have

$$M' - M = dM = Sdx, \quad \text{or} \quad S = \frac{dM}{dx}. \quad . \quad . \quad . \quad . \quad . \qquad (11)$$

That is to say, *the total shear on any transverse section of a beam is equal to the first differential coefficient of the bending moment.*

It follows from this that—

(1) *Where the bending moment is constant the shear is zero.*

(2) *Where the shear is zero the bending moment is at a maximum or a minimum.*

(*b*) *The Distribution of the Shearing Stress across any Transverse Section.*—In Fig. 31 take two transverse sections, dx apart, as before, on which the moments are M and M' respectively. By eq. (5), Art. 26, we have for the stresses on the outer fibres at these two sections

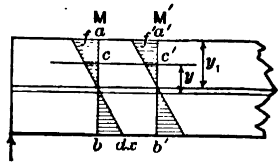

FIG. 31.

$$f = \frac{My_1}{I} \quad \text{and} \quad f' = \frac{M'y_1}{I}.$$

Also for any horizontal section, as cc', the fibre stresses will be

$$p = f\frac{y}{y_1} = \frac{My}{I} \quad \text{and} \quad p' = \frac{M'y'}{I}.$$

The breadth of the beam must be regarded as variable to obtain a general solution, and it will be denoted by b, a variable quantity.

Now the total shearing stress on the section cc', whose area is $b'dx$, is the difference between the total direct stress on $a'c'$ and on ac. But the stress on ac is

$$\int_{y'}^{y_1} pbdy = \int_{y'}^{y_1} \frac{bMydy}{I}.$$

Similarly, the total direct stress on $a'c'$ is

$$\int_{y'}^{y_1} p'bdy = \int_{y'}^{y_1} \frac{bM'ydy}{I}.$$

The difference is

$$\int_{y'}^{y_1} \frac{b(M'-M)ydy}{I} = \int_{y'}^{y_1} \frac{bdMydy}{I}.$$

But $dM = Sdx$ by the previous article, hence we have at last

$$\textit{total stress on plane } cc' = \int_{y'}^{y_1} \frac{bSdxydy}{I} \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

But the area of this section on cc' is bdx. Hence the intensity of the stress on this plane is $\displaystyle\int_{y'}^{y_1} \frac{bSdxydy}{b'Idx}$.

Now S, I, and b' are constant for any given beam, transverse section, and plane of shear cc'; hence these quantities can be removed from under the integral sign, and we have

Intensity of shearing stress at any point in a beam, distant y from the neutral axis, is

$$q' = \frac{S}{Ib'}\int_{y'}^{y_1} bydy. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

Now $\int_{y'}^{y_1} bydy$ is the statical moment of that portion of the cross-section of the beam outside the line cc' on which the shearing stress is obtained, taken about the neutral axis; hence we may say:

The intensity of the shearing stress at any point in the cross-section of a beam is equal to the total shearing stress on that cross-section, multiplied by the statical moment of the area of that portion of the cross-section outside the longitudinal plane of shear in question, about its axis in the neu-

tral plane, divided by the product of the moment of inertia of the entire cross-section into the breadth of the section at that point.

This applies to solid sections of beams of all possible shapes.

For a solid rectangular section b is constant and $\int_y^{y_1} bydy = \frac{b}{2}(y_1^2 - y^2)$.
Hence for this case, where $h = 2y_1$, and $b = b'$ (a constant), we have

$$q' = \frac{6S}{bh^3}\left[\left(\frac{h}{2}\right)^2 - y^2\right] = \frac{3}{2}\frac{S}{bh^3}(h^2 - 4y^2). \quad . \quad . \quad . \quad . \quad (14)$$

Hence the shear at the extreme sides, where $y = \frac{h}{2}$, is zero, and at the neutral axis

$$q_0 = \frac{3}{2}\frac{S}{bh}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

That is to say, *the maximum intensity of the shearing stress on a solid rectangular section is $\frac{3}{2}$ of the mean intensity.*

It is evident, also, that eq. (14) is the equation of a parabola with its vertex on the neutral axis, which is also the axis of the curve. On any particular longitudinal plane, also, the shearing intensity varies from end to end of the beam, as the total shearing stresses on the cross-sections vary, as shown in Fig. 32. By applying equation (14) to various forms of cross-section it can be shown that—

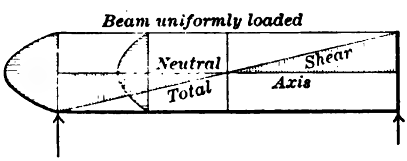

FIG. 32.

1. The maximum shearing intensity in a beam of a solid rectangular section = $\frac{3}{2}$ the mean shear.

2. For a solid circular section it is $\frac{4}{3}$ the mean shear.

3. For I beams and plate girders it is practically equal to the total shear divided by the area of the web portion alone.*

38. To Dimension the Cross-section of a Beam.—(*a*) *For Direct Stress on the Outer Fibre.*—If the beam be of a solid rectangular form of cross-section, use eq. (6). If of any other form, use eq. (5) and evaluate $\frac{I}{y_1}$ by the accompanying table, if the form be one there given. If not, it will be necessary to compute the moment of inertia. If the form be irregular or unsymmetrical, it may be best to obtain the moment of inertia graphically.† In the case of unsymmetrical sections the neutral axis lies at different distances from the outer fibres on the tension and compression sides, and it may be necessary to compute both of them. Since $f_1 = \frac{My_1}{I}$, it is evident these

* See *Modern Framed Structures*, Art. 130.

† *Ibid.*, Art. 127.

stresses per square inch are to each other directly as the distances of their fibres from the neutral axis. Thus in the case of a cast-iron beam the cross-section is made larger on the tension side, as in Fig. 33. Here the outer fibres on the compression side are much farther away from the neutral axis than the outer fibres on the tension side, and hence the maximum stress in compression is much greater than it is in tension, which is as it should be.

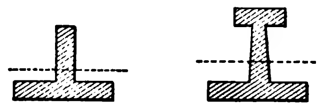

FIG. 33.

For a solid rectangular section we have

$$M = M_0 = \frac{fbh^2}{6}, \quad \text{or} \quad bh^2 = \frac{6M}{f}. \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Take $M =$ maximum bending moment on the beam, in inch-pounds, and $f =$ working value of the stress on the outer fibre. This gives the value of bh^2, and b and h can now be chosen at pleasure, conditioned on bh^2 being equal to the right-hand side of the equation.

(*b*) *For Shearing Stress along the Neutral Axis.*—Since timber is very weak in shearing, as compared to its strength in tension and compression, timber beams and joists of ordinary lengths will usually fail by shearing, and hence they should be dimensioned to safely withstand this shearing action. Lanza shows* that the shearing strength of spruce and white- and yellow-pine beams is about $\frac{1}{30}$ of the transverse modulus at rupture, but he recommends a much smaller factor of safety for shearing than for transverse rupture. If the factor of safety for shearing be two thirds that for transverse strength, we would have the working stress in shearing $\frac{1}{20}$ that in cross-breaking. In order to show what length of wooden beams would require dimensioning for shearing, using this relation of working stresses, we have

$$f = 20q_0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

For a beam loaded at the centre

$$S = \frac{W}{2} \quad \text{and} \quad M = \frac{Wl}{4}.$$

For a beam uniformly loaded

$$S = \frac{W}{2} \quad \text{and} \quad M = \frac{Wl}{8}.$$

Also from (16), for cross-breaking,

$$bh^2 = \frac{6M}{f},$$

and from (15)

$$q_0 = \frac{3}{2}\frac{S}{bh}, \quad \text{or} \quad bh = \frac{3}{4}\frac{W}{q_0}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

* *Applied Mechanics*, 4th ed., p. 696.

For a beam loaded at the centre

$$bh^2 = \frac{6M}{f} = \frac{3\,Wl}{2f}. \quad \ldots \ldots \ldots \quad (19)$$

For a beam uniformly loaded

$$bh^2 = \frac{6M}{f} = \frac{3}{4}\frac{Wl}{f}. \quad \ldots \ldots \ldots \quad (20)$$

From (17), (18), and (19) we find, for beams loaded at the centre, they are equally strong in shearing and in cross-breaking when

$$\frac{l}{h} = \frac{1}{2}\frac{f}{q_0}, \quad \ldots \ldots \ldots \quad (21)$$

and for beams uniformly loaded they are equally strong in these two ways when

$$\frac{l}{h} = \frac{f}{q_0}. \quad \ldots \ldots \ldots \quad (22)$$

For shorter lengths wooden beams are weaker in shearing than in cross-breaking. Hence we have the following

PROPOSITIONS.

Wooden Beams in Shearing and Cross-breaking.

I. *For a centre load the beam should be dimensioned for a shearing stress when the ratio of length to height is less than one half the ratio of the cross-breaking to the shearing working stress.*

II. *For a uniformly distributed load the beam should be dimensioned for a shearing stress when the ratio of length to height is less than the ratio of the cross-breaking to the shearing working stress.*

Thus, for white- and yellow-pine and spruce beams we may take

$$\frac{f}{q_0} = 20.^* \quad \ldots \ldots \ldots \quad (23)$$

Whence

All pine and spruce beams should be dimensioned for shearing failure:

$$\left.\begin{array}{ll} \textit{For concentrated centre load when} & \dfrac{l}{h} \leqq 10. \\ \textit{For uniformly distributed load when} & \dfrac{l}{h} \leqq 20. \end{array}\right\} \quad \ldots \quad (24)$$

In dimensioning for cross-breaking use equations (19) and (20), and for shearing use equation (18), for both concentrated and distributed loads. The following *working values* of f and q_0 may be used.

* Here q_0 is not the true shearing resistance of sound timber, but the shearing resistance of large beams along their neutral axis, where they are usually season-checked.

Species.	Working Values of Cross-breaking Modulus in Pounds per Square Inch. (f)	Working Values of Shearing Modulus in Pounds per Square Inch. (q_0)
White pine....	1000	50
Long-leaf Southern yellow pine	1600	80
Short-leaf Southern yellow pine	1400	70
Norway pine.................	1000	50
Spruce......................	1200	60
White oak....................	1200	100
Red cypress..................	1200	90
Oregon fir....................	1000	50

Tables of working loads for beams of all these species of different lengths and depths are found in Chapter XXXII.

DEFLECTION OF BEAMS.

39. Development of General Formulæ.—Let Fig. 34 represent that portion of a bent beam which is tangent to a horizontal line, the beam being bent under the action of vertical forces. Take the origin on the neutral axis where it becomes horizontal, in the section KL. Then any other

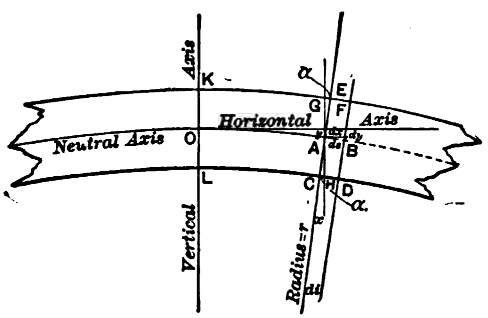

FIG. 34.

section, as EC, distant x from KL, and originally parallel to it, now makes an angle with it which we will call i. These two planes would, therefore, intersect if prolonged, and the radius of the curve of the neutral axis OAB will be called r. Evidently the position of the neutral axis in the plane EC is somewhat below the axis of abscissæ, and the coordinate of this point A is now $+x$ and $-y$, with reference to the origin O.

If we now draw the line GH parallel to KL, the intercepts in the outer fibres, between this section and EC, are the distortions of these fibres in the length $OA \doteq x$. This distortion may be called α.

To investigate the law of the relative changes in x and y, take another section, FD, distant dx from EC. Then the coordinates of B with reference to A are $+dx$ and $-dy$, and the actual length along the neutral axis from A to B is $ds = \sqrt{dx^2 + dy^2}$. Also the angle between EC and FD is di.

The angle i is also the angle the neutral axis at A forms with the horizontal, or

$$i = \frac{dy}{dx} \quad \text{and} \quad di = \frac{d^2y}{dx}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

But di is also equal to $\frac{ds}{r}$;

$$\therefore \quad \frac{1}{r} = \frac{d^2y}{dxds}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

Evidently, when the deflection angle i is small, dx is practically equal to ds, in which case $dxds = dx^2$; whence eq. (23) becomes

$$\frac{1}{r} = \frac{d^2y}{dx^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

We also have $C\dot{A}H = GAB = i = \frac{\alpha}{AE} = \frac{\alpha}{y_1}$, where y_1 is the distance from the neutral axis to the outer fibre. Also

$$i = \frac{x}{r} = \frac{\alpha}{y_1} \quad \text{and} \quad \frac{1}{r} = \frac{\alpha}{xy_1}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (25)$$

But from eq. (2), Chapter I, we have $E = \frac{pl}{\alpha}$, or $\alpha = \frac{pl}{E}$. In this case $l = x$, and $p = f =$ stress on outer fibre; hence we have $\alpha = \frac{fx}{E}$.

Also from eq. (5) we have $M_0 = \frac{fI}{y_1}$, or $y_1 = \frac{fI}{M_0}$.

Substituting these values of α and y_1 in eq. (25), and also M for M_0, we obtain

$$\frac{1}{r} = \frac{\alpha}{xy_1} = \frac{fx}{E} \cdot \frac{1}{x} \cdot \frac{M}{fI} = \frac{M}{EI}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

Hence we have

$$\frac{1}{r} = \frac{M}{EI} = \frac{d^2y}{dx^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

These are the fundamental equations for use in all problems in the deflection of beams.*

The relation utilized to find deflection angles and movements is

$$\frac{d^2y}{dx^2} = \frac{M}{EI}, \quad \text{or} \quad d\left(\frac{dy}{dx}\right) = \frac{Mdx}{EI}.$$

* It is here assumed that the deflection is due wholly to the longitudinal deformation of the fibres from bending moment, and not at all from the action of the shearing forces, which is substantially correct when $\frac{l}{h}$ is large (see Art. 46).

That is to say, *the change in the angle, or the amount of bending effected over a certain length of the beam, is equal to the bending moment into this length divided by EI.* To find the total angle or total amount of bending in a given finite distance, therefore, where the bending moment is usually changing at all points, it is necessary to integrate, or sum up, the infinitesimal changes between certain limits or between certain transverse sections. Then having found the law of the curvature of the beam, the deflection, or vertical displacement at any point, can be found by integrating again from y' to y between the same transverse sections of the beam. The only difficulty in this work is encountered in finding the constants of the first integration. The following cases are the most common, in which both E and I are taken as constant throughout the length of the beam. Bending moment producing convexity downwards is called positive.

40. Beam Fixed at One End and Loaded at the Other.—Here we have, for the value of the bending moment at any section x, $M_x = -P(l-x)$; hence

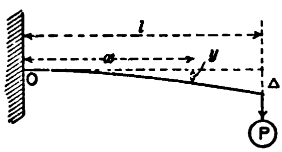

FIG. 35.

$$\frac{d^2y}{dx} = d\left(\frac{dy}{dx}\right) = di = -\frac{P}{EI}(l-x)dx.$$

Integrating this between the limits 0 and x, we have

$$\frac{dy}{dx} = i = -\frac{P}{EI}\left(lx - \frac{x^2}{2}\right). \quad [+C = 0, \text{ since, for } x = 0, \frac{dy}{dx} = 0.]$$

Integrating again from 0 to x, we have, as the deflection at any section,

$$y = -\frac{P}{EI}\left(\frac{lx^2}{2} - \frac{x^3}{6}\right). \quad [+C = 0, \text{ since, for } x = 0, y = 0.]$$

To find the maximum angle, and also maximum deflection, make $x = l$, and obtain

$$\max. i = -\frac{Pl^2}{2EI}; \quad \dots\dots\dots \quad (28)$$

$$\max. y = \Delta = -\frac{Pl^3}{3EI}. \quad \dots\dots\dots \quad (29)$$

41. Beam Fixed at One End and Uniformly Loaded.—Let the load per unit of length $= p$. Then the bending moment at any section x is $M_x = -\frac{p(l-x)^2}{2}$; hence we have

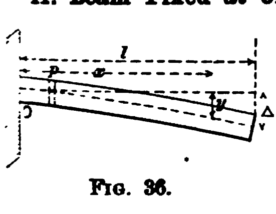

FIG. 36.

$$\frac{d^2y}{dx} = d\left(\frac{dy}{dx}\right) = di = -\frac{p}{2EI}(l^2 - 2lx + x^2)dx.$$

Integrating this from 0 to x, we have

$$\frac{dy}{dx} = i = -\frac{p}{2EI}\left(l^2x - lx^2 + \frac{x^3}{3}\right). \quad [+C = 0, \text{ since, for } x = 0, \frac{dy}{dx} = 0.]$$

Integrating again from 0 to x, we obtain, as the deflection at any section,

$$y = -\frac{p}{2EI}\left(\frac{l^2x^2}{2} - \frac{lx^3}{3} + \frac{x^4}{12}\right). \quad [+ C = 0, \text{ since, for } x = 0, y = 0.]$$

Again, we find the maximum angle and deflection for $x = l$, where

$$\text{max. } i = -\frac{pl^3}{6EI}; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

$$\text{max. } y = \Delta = \frac{pl^4}{8EI}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

42. Beam Supported at the Ends and Loaded at the Centre.—Let the load P be placed at the centre. Then the moment at any section is

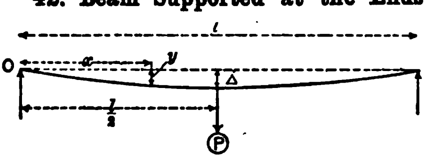

FIG. 37.

$$M_x = \frac{Px}{2}; \text{ hence we have}$$

$$\frac{d^2y}{dx} = di = \frac{Pxdx}{2}.$$

Integrating this from 0 to x, we have

$$\frac{dy}{dx} = i = \frac{Px^2}{4} + C.$$

Now C is the value of the angle i at the origin, where $x = 0$, or at the lower limit of the integration; in other words, C is the value of the angle we started with, and we must add to this, algebraically, the *changes* of the angle from 0 to x. To find the value of this constant we must pass to a point where the value of the angle is known. In this case we know this angle is zero for $x = \frac{l}{2}$. Hence make $x = \frac{l}{2}$, for which $\frac{dy}{dx} = 0$, and we have $C = -\frac{Pl^2}{16}$. Therefore

$$\frac{dy}{dx} = i = \frac{Px^2}{4} - \frac{Pl^2}{16} = \frac{P}{4}\left(x^2 - \frac{l^2}{4}\right).$$

Integrating again, we have

$$y = \frac{P}{4}\left(\frac{x^3}{3} - \frac{l^2x}{4}\right). \quad [+ C = 0, \text{ since, for } x = 0, C = 0.]$$

The maximum value of i is evidently at the ends, and of y at the centre. Hence we have

$$\text{max. } i = -\frac{Pl^2}{16}; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

$$\text{max. } y = \Delta = \frac{Pl^3}{48EI}.^* \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$$

---

* For this case the deflection due to shear is $\frac{Pl}{4E_sA}$. (See Art. 46.)

43. Beam Supported at the Ends and Uniformly Loaded.—Let the load per unit of length $= p$. Then the bending moment at any section x is

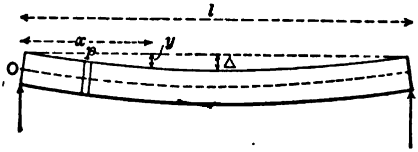

Fig. 38.

$$M_x = \frac{plx}{2} - \frac{px^2}{2} = \frac{p}{2}(lx - x^2);$$

$$\frac{d^2y}{dx} = di = \frac{p}{2EI}(lx - x^2)dx.$$

Integrating once, we have

$$\frac{dy}{dx} = i = \frac{p}{2EI}\left(\frac{lx^2}{2} - \frac{x^3}{3}\right) + C.$$

As before, for $x = \frac{l}{2}$, $\frac{dy}{dx} = 0$; hence $C = -\frac{p}{2EI}\left(\frac{l^3}{12}\right)$;

$$\therefore \frac{dy}{dx} = i = \frac{p}{2EI}\left(\frac{lx^2}{2} - \frac{x^3}{3} - \frac{l^3}{12}\right).$$

Integrating again, we have

$$y = \frac{p}{2EI}\left(\frac{lx^3}{6} - \frac{x^4}{12} - \frac{l^3x}{12}\right). \quad [+ C = 0, \text{ since, for } x = 0,\ y = 0.]$$

Max. i is found for $x = 0$, and max. y for $x = \frac{l}{2}$. Hence

$$\text{max. } i = -\frac{pl^3}{24EI}. \quad \ldots\ldots\ldots \quad (34)$$

$$\text{max. } y = \Delta = -\frac{5}{384}\cdot\frac{pl^4}{EI}*. \quad \ldots\ldots\ldots \quad (35)$$

44. More Complicated Cases† are such as—

(*a*) Beam supported at the ends and loaded at any point.

(*b*) Fixed at one end and loaded in any manner.

(*c*) Fixed at both ends and loaded in any manner.

These and other cases are treated in works on Applied Mechanics, and they will not be further considered here. The difficulty in such cases is to evaluate the constants of integration. While this can always be done, the algebraic reductions are long and tedious. The following table gives all the results for the ordinary cases which are usually needed in practice.

45. Table of Moments, Stresses, and Deflections of Beams having Constant Moments of Inertia.

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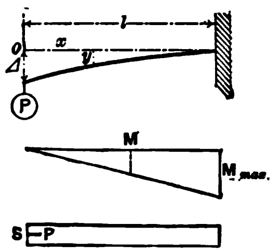
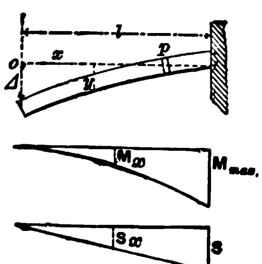
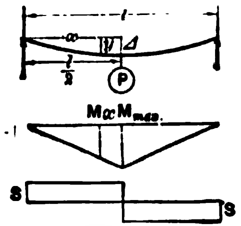
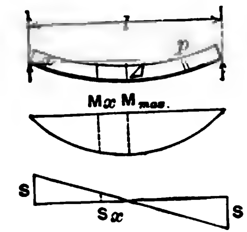
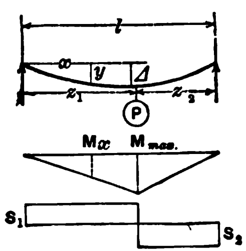
* For this case the deflection from shear is $\frac{pl^2}{8E_sA}$. (See Art. 46.)

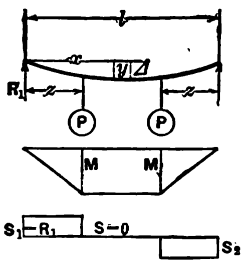
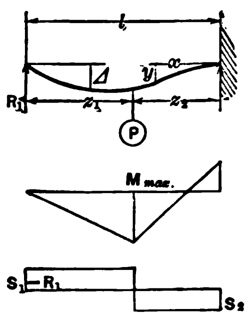
† For an excellent design of a home-made apparatus to be used in testing the correctness of all kinds of beam formulæ, see a paper on this subject by Prof. James L. Greenleaf in *Jour. Franklin Inst.* for July 1895, vol. CXL. p. 27.

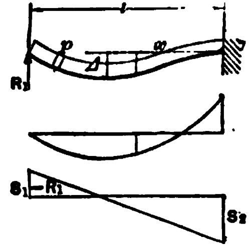
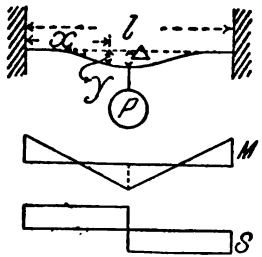
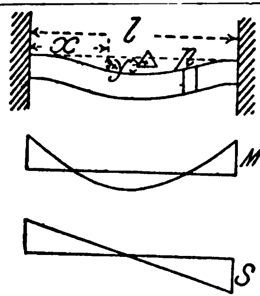
MOMENTS, STRESSES, AND DEFLECTION OF BEAMS.

The Beam and its Load with the Moment and Shear Diagrams.	Moment Equation and Maximum Moment. M_x and M_{max}.	Equation of Elastic Line, and Maximum Deflection in terms of the Loading. y and Δ_w	Maximum Deflection in terms of Stress on Extreme Fibre of Symmetrical Sections. Δ_f	Maximum Stress on Extreme Fibre in terms of the Loading. Symmetrical Sections. f.
 Fig. 39.	$M_x = -Px$ $M_{max.} = -Pl$	$y = \frac{P}{6EI}[2l^3 - 3l^2x + x^3]$ $\Delta = \frac{Pl^3}{3EI}$	$\frac{2fl^2}{3Eh}$	$\frac{Plh}{2I}$
 Fig. 40.	$M_x = -\frac{Px^2}{2}$ $M_{max.} = -\frac{pl^2}{2}$	$y = \frac{p}{24EI}[x^4 - 4l^3x + 3l^4]$ $\Delta = \frac{pl^4}{8EI}$	$\frac{fl^2}{2Eh}$	$\frac{pl^2h}{4I}$

 FIG. 41.	$M_x = \frac{P}{2}x$ $M_{max.} = \frac{Pl}{4}$	$y = \frac{Px}{48EI}[3l^2 - 4x^2]$ $\Delta = \frac{Pl^3}{48EI}$	$\frac{fl^2}{6Eh}$	$\frac{Plh}{8I}$
 FIG. 42.	$M_x = \frac{px}{2}(l - x)$ $M_{max.} = \frac{Pl^2}{8}$	$y = \frac{px}{24EI}[l^3 - 2lx^2 + x^3]$ $\Delta = \frac{5pl^4}{384EI}$	$\frac{5}{24}\frac{fl^2}{Eh}$	$\frac{pl^3h}{16I}$
 FIG. 43.	$x < z_1$ $M_x = \frac{Pz_2x}{l}$ $x > z_1$ $M_x = \frac{Pz_2x}{l} - P(x - z_1)$ $M_{max.} = \frac{Pz_1z_2}{l}$	$x < z_1$ $y = \frac{Pz_2x}{6EIl}[2lz_1 - z_1^2 - x^2]$ $x > z_1$ $y = \frac{Pz_1(l - x)}{6EIl}[2lx - x^2 - z_1^2]$ $\Delta = \frac{Pz_2}{27EIl}\sqrt{3[z_1(2z_2 + z_1)]^3}$ for $x = \frac{1}{3}\sqrt{3[z_1(2z_2 + z_1)]}$	$\frac{f}{27Eh}\sqrt{3z_1(2z_2 + z_1)^3}$	$\frac{Pz_1z_2h}{2Il}$

MOMENTS, STRESSES, AND DEFLECTION OF BEAMS.

The Beam and its Load with the Moment and Shear Diagrams.	Moment Equation and Maximum Moment. M_x and M_{max}.	Equation of Elastic Line, and Maximum Deflection in terms of the Loading. y and Δ_w.	Maximum Deflection in terms of Stress on Extreme Fibre of Symmetrical Sections. Δ_f.	Maximum Stress on Extreme Fibre in terms of the Loading, Symmetrical Sections. f.
 FIG. 44.	$x < z$ $M_x = Px$ $x > z$ $M = Pz = M_{max.}$	$x < z$ $y = \frac{Px}{6EI}[3lz - 3z^2 - x^2]$ $x > z$ $y = \frac{Pz}{6EI}[3lx - 3x^2 - z^2]$ $\Delta = \frac{Pz}{6EI}\left[\frac{3}{4}l^2 - z^2\right]$	$\frac{f}{3Eh}\left[\frac{3}{4}l^2 - z^2\right]$	$\frac{Pzh}{2I}$
 FIG. 45.	$R_1 = \frac{P}{2l^3}[3lz_2^2 - z_2^3]$ $x < z_2$ $M_x = R_1(l - x) - P(z_2 - x)$ $x > z_2$ $M_x = R_1(l - x)$ $M_{max.} = R_1(l - z_2)$ for $x = z_2$	$x < z_2$ $y = \frac{1}{6EI}[R_1x^3 - 3R_1lx_2 + 3Pz_2x^2 - Px^3]$ $x > z_2$ $y = \frac{1}{6EI}[R_1x_1^3 - 3R_1lx^2 + 3Pz_2^2x - Pz_2^3]$ $\Delta = \frac{Pz_2^2}{6EI}(l - z_2)\sqrt{\frac{l - z_2}{3l - z_2}}$ for $x = l\left(1 - \sqrt{\frac{l - z_2}{3l - z_2}}\right)$	$\frac{2f}{3Eh}\sqrt{(3l - z_2)(l - z_2)}$	$\frac{Ph}{4l^3I}(3lz_2^2 - z_2^3)(l - x)$

 FIG. 46.	$R_1 = \frac{3}{8}pl$ $M_x = \frac{p}{4}(4x - l)(l - x)$ $M_{max.} = -\frac{pl^2}{8}$ for $x = 0$	$y = \frac{px^2}{48EI}(l - x)(3l - 2x)$ $\Delta = 0.0054\frac{pl^4}{EI}$ for $x = 0.578l$	$\frac{0.0864fl^2}{Eh}$	$\frac{pl^2h}{16I}$
 FIG. 47	$M_x = \frac{Px}{2} - \frac{Pl}{8}$ $M_{max.} = \frac{Pl}{8}$ for $x = 0$ and $x = \frac{l}{2}$	$y = \frac{P}{8EI}\left(\frac{2x^3}{3} - \frac{lx^2}{2}\right)$ $\Delta = \frac{Pl^3}{192EI}$	$\frac{fl^2}{12Eh}$	$\frac{Plh}{16I}$
 FIG. 48.	$M_x = \frac{p}{2}(lx - x^2) - \frac{pl}{12}$ $M_{max.} = \frac{pl^2}{12}$ for $x = 0$	$y = \frac{p}{12EI}\left(lx^3 - \frac{x^4}{2} - \frac{l^2x^2}{2}\right)$ $\Delta = \frac{pl^4}{384EI}$	$\frac{fl^2}{16Eh}$	$\frac{pl^2h}{24I}$

46. Deflections from Shearing Forces.—For short beams it is necessary to take into account the shearing forces also. Since the modulus of elasticity in shearing is the ratio of the shearing stress to the angular distortion (transverse distortion per unit of length, since the angle is equal to its tangent), we may say that for a distance along the beam of dx where the shearing stress per unit area is $s = \frac{S}{A}$*, the differential deflection from shear is

$$dy_s = \frac{s}{E_s}dx = \frac{S}{E_sA}dx. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

To integrate this we have to express S and A as functions of x. The cross-section, A, will be assumed as constant, and for a concentrated load S is also constant and equal to the supporting force on that side of the load. For a uniformly distributed load S is equal to the algebraic sum of the forces on one side of the plane of shear, which here must be taken normal to the deflection. Thus for any section distant x from the end of the beam we have for a concentrated load at the centre $S = \frac{P}{2}$, a constant, while for a beam uniformly loaded (supported at the ends in both cases) $S = \frac{P}{2} - px = p\left(\frac{l}{2} - x\right)$.

Using these values in eq. (36), we have for the

Deflection of a beam from shear when supported at its ends and loaded at the centre,

$$Y_s = \frac{P}{2E_sA}x, \text{ or at centre } \Delta_s = \frac{Pl}{4E_sA}. \quad . \quad . \quad . \quad . \quad (37)$$

Deflection of a beam from shear when supported at its ends and uniformly loaded,

$$dY_s = \frac{p}{E_sA}\left(\frac{l}{2} - x\right)dx,$$

or

$$y_s = \frac{p}{E_sA}\left(\frac{lx}{2} - \frac{x^2}{2}\right), \text{ or at centre } \Delta_s = \frac{pl^2}{8E_sA}. \quad . \quad . \quad . \quad (38)$$

Hence the *total deflections at the centre* for these two cases are

$$\Delta = P\left(\frac{l^3}{48EI} + \frac{l}{4E_sA}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

and

$$\Delta = p\left(\frac{5l^4}{384EI} + \frac{l^2}{8E_sA}\right), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (40)$$

* Assuming also that the shearing stress is uniformly distributed over the cross-section.

for beams supported at the ends and loaded with a single concentrated load P at the centre, and with a uniformly distributed load of p per unit of length, respectively.

For the metals take $E_s = \frac{2}{5}E$, while for wood take $E_s = \frac{1}{5}E$. The fibrous character of wood may explain the apparent anomaly.

For solid rectangular wooden beams, therefore, we have

For load at centre, from (39), making $E_s = \frac{1}{5}E$, and $\frac{1}{A} = \frac{r^2}{I} = \frac{1}{12}\frac{h^2}{I}$,

$$\Delta = \frac{Pl}{48EI}(l^2 + 5h^2), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (41)$$

and

For beam uniformly loaded, from (40),

$$\Delta = \frac{5pl^2}{96EI}\left(\frac{l^2}{4} + h^2\right). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (42)$$

These equations show that when a rectangular wooden beam loaded at the centre has a length less than seven times the height, the deflection from shear is more than ten per cent of the total deflection, while for such a beam uniformly loaded the deflection from shear exceeds ten per cent of the total when the length is less than about six times the height.

47. Determination of Young's Modulus of Elasticity from Bending Tests.—Since E enters in all the expressions for deflection of beams, it is evident that it may be found from a bending test where all the dimensions, loads, and deflections are observed. Thus for a beam of uniform, solid, rectangular cross-section, supported at the ends and loaded at the centre, we should have, from eq. (33), for a long beam where deflection from shearing forces could be neglected,

$$E = \frac{Pl_1^3}{48\Delta I} = \frac{Pl^3}{4\Delta bh^3} = \frac{fl^2}{6\Delta h}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (43)$$

Since in testing a beam the stress on the extreme fibre is also desired, the last form of this equation may be useful in case f is also to be computed. However, this value of f must be *inside the elastic limit* in order to use it in computing E.

It is best to measure a series of coincident loads and deflections, and plot them as in Fig. 49, then draw a tangent to the curve at the origin and use this in finding E. Thus the tangent line OA is used for computing E, and *the coordinates of any point on this line may be taken.* It is convenient to take a point representing a deflection of unity. On this curve this corresponds to a load of 6250. The dimensions of the beam were $l = 140$ in., $b = 4.02$ in., $h = 8.04$ in., and the material was long-leaf

yellow pine (*Pinus palustris*). Using the second form of eq. (43), we have

$$E = \frac{Pl^3}{4\Delta bh^3} = \frac{6250 \times 140^3}{4 \times 1 \times 4.02 \times 8.04^3} = 2{,}070{,}000 \text{ pounds per square inch.}$$

The maximum load was 13,500 pounds, from which we find, by eq. (6), *the computed* maximum stress on the outer fibre to be

$$f = \frac{6m}{bh^2} = \frac{3}{2}\frac{Wl}{bh^2} = 10{,}000 \text{ pounds per square inch.}$$

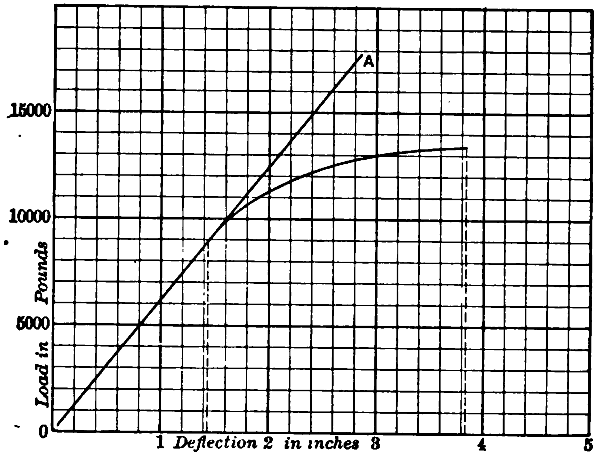


FIG. 49.

The elastic-limit load might be taken as 9000 pounds, whence the fibre-stress at this limit would be

$$f = \frac{3}{2}\frac{W'l}{bh^2} = \frac{3 \times 9000 \times 140}{2 \times 4.02 \times 8.04^2} = 7300 \text{ lbs. per square inch.}$$

48. The Rational Designing of Flitched Beams.*—A flitched beam is one composed of two sticks of timber enclosing between them a wrought-iron or steel plate of the full length of the sticks, these three members being rigidly bolted together, preferably along the neutral plane, in such a way that they will act as one solid member when deflecting under a load. In order that these two materials may come to their working stresses simultaneously, the iron or steel plate should always be of less depth than that of the timber.

To find the relative depths of steel (or wrought iron) plate and the timber sides in order that they shall come to their working stresses at the

* This problem is introduced here, not because it is very common or important in itself, but because it is a good type of composite systems and illustrates the method of analyzing such systems.

same time, we must utilize the principle that *when two or more members jointly carry a single load, they share this load in direct proportion to their relative rigidities.* The rigidity of a beam is the inverse of its flexibility, and the flexibility is measured by the deflection under a given load. Hence the rigidity will be measured by the reciprocal of the deflection. The equation representing the deflection of a solid rectangular beam, in terms of the stress on the outer fibre, is, from eq. (33), since

$$M = \frac{1}{6}fbh^2 = \frac{Pl}{4} \text{ for a load } P \text{ at the centre.}$$

$$\Delta = \frac{Pl^3}{48EI} = \frac{fl^2bh^2}{72EI} = \frac{fl^2}{6Eh}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (44)$$

But since the rigidity is measured by the reciprocal of the deflection, we have as a measure of the rigidity of a rectangular beam, in terms of the stress on the outer fibre,

$$R = \text{rigidity} = \frac{1}{\Delta} = \frac{6Eh}{fl^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (45)$$

We may now write the proportion:

Deflection of the wooden beam : deflection of the steel plate :: the rigidity of the plate : the rigidity of the beam,

or

$$\Delta_w : \Delta_s :: R_s : R_w, \text{ or } \frac{\Delta_w}{\Delta_s} = \frac{R_s}{R_w}. \quad . \quad . \quad . \quad . \quad . \quad (46)$$

But

$$\frac{R_s}{R_w} = \frac{6E_s h_s}{f_s l^2} \div \frac{6E_w h_w}{f_w l^2} = \frac{E_s h_s f_w}{E_w h_w f_s}. \quad . \quad . \quad . \quad . \quad . \quad (47)$$

Hence we have for a flitched beam, in which $\Delta_w = \Delta_s$,

$$\frac{\Delta_w}{\Delta_s} = \frac{R_s}{R_w} = \frac{E_s h_s f_w}{E_w h_w f_s} = 1, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (48)$$

where R_w = rigidity of the timber sides;
R_s = same for the steel plate;
Δ_w = deflection of the timber sides;
Δ_s = same of the steel plate;
E_w = modulus of elasticity of timber = from 1,000,000 in white pine to 1,800,000 in long-leaf yellow pine;
E_s = modulus of elasticity of wrought iron and steel = 28,000,000;
P = total load on flitched beam;
P_w = load carried by the timber sides;
P_s = same for the steel plate;
f_w = working fibre-stress for timber = from 1000 in white pine to 1800 in long-leaf yellow pine;
f_s = same for steel = 12,000 to 18,000 pounds per square inch;

h_w = depth of the timbers in inches;
h_s = same for the steel plate;
b_w = total thickness of both timbers in inches;
b_s = same for the steel plate.

From eq. (48) we may derive many important relations:

(*a*) *To find the relative depths of steel plate and wooden beams to give simultaneous working stresses in each.* Eq. (48) may be written

$$\frac{h_w}{h_s} = \frac{E_s f_w}{E_w f_s}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (49)$$

Example: Let $E_s = 28{,}000{,}000$, $E_w = 1{,}400{,}000$, $f_s = 16{,}000$, $f_w = 1600$; then

$$\frac{h_w}{h_s} = \frac{28{,}000{,}000 \times 1600}{1{,}400{,}000 \times 16{,}000} = 2.$$

That is to say, the wooden sides must be twice as deep as the steel plate, regardless of their respective thicknesses, in order to give a working stress in the wooden sides of one tenth that in the steel plate.

(*b*) *To find the relative stresses on the outer fibres when the plate is of the full depth of the timber sides.* We now put eq. (48) in the form

$$\frac{f_w}{f_s} = \frac{E_w h_w}{E_s h_s}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (50)$$

Using the same values of E as above, and making $h_w = h_s$, we have

$$\frac{f_w}{f_s} = \frac{E_w}{E_s} = \frac{1}{20}.$$

Hence when the steel or iron plate has the full depth of the wooden sides, the stress in the outer fibres of the timber is only one twentieth that in the plate. This does pretty well for a white-pine and steel combination.

But in the case of white pine we should not take E_w higher than 1,000,000. Hence we have for white pine and steel of equal depths

$$\frac{f_w}{f_s} = \frac{E_w}{E_s} = \frac{1{,}000{,}000}{28{,}000{,}000} = \frac{1}{28}.$$

That is, the maximum stress in the timber is only $\frac{1}{28}$ that in the steel plate. For an elastic limit of steel of 40,000 pounds per square inch we may have a working fibre-stress of 20,000 pounds per square inch. This would give a fibre-stress of 700 pounds per square inch in the timber sides, which is hardly a sufficiently high working stress for white pine. *All these conclusions are quite independent of the relative thicknesses of plate and sides.*

To find what part of the total load P is carried by the timber sides and by the steel plate respectively, we may let P_w and P_s represent these loads, so that $P_w + P_s = P$. Also the total load P divides itself between the parts in proportion to their respective rigidities, these rigidities being now

taken as the reciprocals of the deflections when expressed in terms of the equal loads W instead of fibre-stresses. From eq. (44) we have

$$R_s = \frac{1}{\Delta_s} = \frac{48E_sI_s}{P_sl^3} \quad \text{and} \quad R_w = \frac{1}{\Delta_w} = \frac{48E_wI_w}{P_wl^3}, \quad . \quad . \quad . \quad (51)$$

whence we have

$$\frac{P_s}{P_w} = \frac{R_s}{R_w} = \frac{E_sI_s}{E_wI_w}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (52)$$

But for solid rectangular sections $I = \frac{1}{12}bh^3$; hence we have

$$\frac{P_s}{P_w} = \frac{E_sb_sh_s^3}{E_wb_wh_w^3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (53)$$

But $P_w = P - P_s$, which substituted in (51) and reduced gives

$$P_s = \frac{P}{1 + \frac{E_wb_wh_w^3}{E_sb_sh_s^3}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (54)$$

Similarly,

$$P_w = \frac{P}{1 + \frac{E_sb_sh_s^3}{E_wb_wh_w^3}} = P - P_s. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (55)$$

Thus if the depths and thicknesses of the plate and of the timber sides be known, the parts of the total load which they will carry can be found from equation (54) or (55), or their relative values may be found at once from equation (53).

EXAMPLE: *Dimension a flitched beam* 24 *feet long to carry a distributed load of* 2000 *lbs. per foot.*

Assume a depth of timber sides of 16 inches, and let the plate be the full depth of the timbers. If we use "long-leaf" pine, we may take $E_w = 1{,}400{,}000$, while $E_s = 28{,}000{,}000$ for the steel plate. Eq. (50) now gives us $\frac{f_w}{f_s} = \frac{1}{20}$. That is, the maximum fibre-stress in the timber sides is one twentieth that in the steel plate. We will also assume the plate to be $\frac{1}{2}$ inch thick. If it is stressed to 20,000 lbs. per square inch, the load it alone will carry is found from eq. (6). Thus

$$M_s = \frac{P_sl}{8} = M_0 = \frac{f_sb_sh_s^2}{6}, \quad \text{or} \quad P_s = 12{,}000 \text{ lbs., nearly.}$$

This leaves 36,000 pounds to be carried by the timber sides.

But when the stress in the plate is 20,000 pounds, that in the timber sides is but 1000 pounds. Hence we must now find the combined breadth of the two sides to carry 36,000 pounds with this fibre-stress. Here again we have

$$M_w = \frac{P_wl}{8} = M_0 = \frac{f_wb_wh_w^2}{6}, \quad \text{or} \quad b_w = \frac{3}{4}\frac{P_wl}{f_wh_w^2} = 30 \text{ inches, nearly.}$$

As this thickness is out of the question, we might double the thickness of the steel plate, making it 1 inch, when it will carry 24,000 pounds, leaving 24,000 pounds for the timber sides. This would reduce them to 20 inches in width, or two sticks, 10 in. by 16 in. each.

If it were practical to obtain timbers 18 inches deep, they would serve the purpose much better. (The student might redimension the beam on this assumption.)

It is evident from the above that there is no economy in combining iron and wood in this manner. An iron or steel I beam or a plate girder should always be used in such a case when this is practicable. The problem has been inserted here as a valuable exercise.

49. Steel and Concrete in Combination.—It is now common to employ steel wires or bars to strengthen the tension sides of concrete beams. To analyze this case it is necessary to know the modulus of elasticity of the particular concrete employed, and at the age when its working strength is first required. This property of concrete has seldom been observed (see Chapter XXX), but for good Portland-cement concrete it may be taken at 1,000,000. For cinder concrete, such as is used in fire-proof flooring in buildings, it is very much less, possibly not over 100,000.

Referring again to the general proposition that in composite structures the load divides itself between the systems in direct proportion to their relative rigidities, we conclude that for like areas, similarly placed, the rigidities are to each other as their moduli of elasticity. Since the modulus of elasticity of steel is 28,000,000 and of the concrete, say, 1,000,000, it follows that one square inch in section of steel resists for equal deformations as much as 28 square inches of concrete similarly placed. To find the resistance of the combined material, therefore, substitute an amount of concrete for the steel wire or bar equal to twenty-eight times its cross-section, adding this in the horizontal plane of the steel bar, and then treat this new form of section, as shown in Fig. 50, as an actual beam of concrete. By finding its moment of inertia, the strength of the beam, when the concrete fails by cracking on the tension side, may be found from the equation $M_o = \frac{fI}{y_1}$, where f is the ultimate tensile strength of the concrete; I is the moment of inertia of the transformed section; y_1 is the distance from the neutral axis of this section to the tension side of the beam; and M_0 is the moment of resistance of the actual beam when the concrete cracks.*

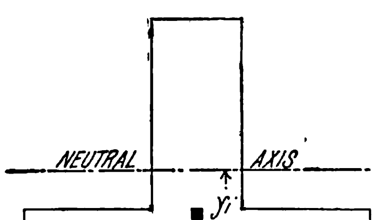


FIG. 50.—Steel and Concrete in Combination.

* For a discussion of the case where the concrete cracks and the elastic limit of the steel bar is reached, as well as for the case where the concrete cracks on the tension side and then fails in compression because of the strength of the steel bar, including also the case here treated, see an article by the author in *Engr. News*, Jan. 3, 1895, p. 10.

If b = breadth of concrete beam,
h = height " " "
a = area of steel bar,
A = substituted equivalent area of concrete $= a\frac{E_s}{E_c}$,
E_s = modulus of elasticity of steel,
E_c = " " " " the concrete used,
e = distance from centre of beam to centre of bar,
d = " " " " " " new neutral axis,
y_1 = " " neutral axis to tension side of beam,
y_2 = " " " " " compression side of beam,
f_t = stress on outer portion of the concrete on the tension side,
f_c = " " " " " " " " " compression side,
I = moment of inertia of the transformed cross-section,
f_0 = stress on outer portion of beam if no steel bar were used $= \frac{6M}{bh^2}$,
$m \doteq \frac{bh}{a} \cdot \frac{E_c}{E_s}$, used for convenience (but is in fact the ratio of the longitudinal rigidity of the beam to that of the steel bar),

then we have

$$d = \frac{l}{m+1}; \quad y_1 = \frac{h}{2} - d; \quad y_2 = \frac{h}{2} + d; \quad I = bh\left(\frac{h^2}{12} + \frac{l^2}{1+m}\right).$$

Tensile stress on concrete at bottom $\Bigg\} = f_t = \frac{My_1}{I} = f_0\left(\frac{1+m-2\frac{e}{h}}{1+m+12\frac{e^2}{h^2}}\right).$ (A)

Compressive stress on concrete at top $\Bigg\} = f_c = \frac{My_2}{I} = f_0\left(\frac{1+m+2\frac{e}{h}}{1+m+12\frac{e^2}{h^2}}\right).$ (B)

If the steel bar be a flat plate and this be placed at the bottom of the beam, but buried in the concrete, then $e = \frac{h}{2}$ and we have

Tensile stress on concrete at bottom $= f_t' = f_0\left(\frac{m}{m+4}\right)$, (A′)

and similarly

Compressive stress on concrete at top $= f_c' = f_0\left(\frac{m+2}{m+4}\right)$. (B′)

If the steel rod, or plate, be removed still farther from the body of the concrete, by placing it in the lower side of a projecting rib of concrete, then e becomes greater than $\frac{h}{2}$. Equations (A) and (B) will still apply to this case, merely using the true values of e and h, not counting the projecting rib as any part of the concrete beam. Thus if a concrete floor 4 inches thick be supported by ribs every two feet, in the bottoms of which are steel rods 1

inch square, so placed as to be 10 inches below the centre of the concrete floor, then from equations (A) and (B) we have

$$f_t = \frac{3.43}{79.43}f_0 = 0.007f_0 \quad\text{and}\quad f_c = \frac{9.43}{79.43}f_0 = 0.119f_0,$$

where $f_0 = \frac{6M}{bh^2} = \frac{M}{384}$.

If any particular ratio of compressive to tensile strength of the concrete is to be developed, we may impose the condition $\frac{f_c}{f_t} = k$; whence for the steel placed at the bottom side of the beam we have, from equations (A′) and (B′),

$$\frac{f_c}{f_t} = k = \frac{m+2}{m}, \quad\text{or}\quad m = \frac{bh}{a}\frac{E_c}{E_s} = \frac{2}{k-1};$$

whence, for $\frac{f_c}{f_t} = k$,

$$a = \frac{bh(k-1)E_c}{2E_s}. \qquad \text{(C)}$$

Thus if $\frac{f_c}{f_t} = 5$, we have, for $\frac{E_c}{E_s} = \frac{1}{28}$, $a = \frac{bh}{14}$.

That is to say, if the steel plate covered the entire base of the beam, it would have to be $\frac{1}{14}$ as thick as the concrete and steel combined to satisfy this condition, it being assumed in this and all former cases that the concrete does not crack on the tension side. Evidently it is impracticable to develop the full compressive strength of the concrete by this construction, on condition that the concrete is to remain unbroken on the tension side.

To find the total stress in the steel bar, we assume it to stretch the same as the parts of the concrete beam adjacent to it; hence for any given position, distant e from the centre of the beam, we have

$$\textit{Total stress on steel bar} = \frac{y_1}{f_1}(e-d)\frac{E_s}{E_c}a = \frac{2f_0eb}{1+m+12\frac{l^2}{h^2}}. \qquad \text{(D)}$$

If $e = \frac{h}{2}$, this becomes, for the bar at the bottom,

$$\textit{Total stress on steel bar at bottom} = \frac{1}{m+4}\left(\frac{6M}{h}\right). \qquad \text{(D′)}$$

For this case the tensile stress in the steel rod, in pounds per square inch is $\frac{E_s}{E_c}f_1$, or it is $\frac{E_s}{E_c}$ times as much as that in the concrete adjoining it. This stress in the steel bar can never be more than from 2000 to 5000 pounds per square inch in rock or gravel concrete, but in cinder concrete it would be very much more. To utilize the strength of the steel, therefore, in rock concrete, it is necessary either to allow the concrete beam to crack on the tension side or to remove the steel bars to the lower portions of projecting ribs.

50. Approximate Determination of the Strength of Flat Plates under Normal Forces.*—(*a*) *Flat Circular Plate Supported at the Circumference and Uniformly Loaded.*—Assume a diametral strip 1 in. in width to be loaded over its full width at the ends, but the loaded surface to reduce to a zero width at the centre, this load to be p lbs. per square inch. The total load on the strip will then be pr, and each end support will be $\frac{pr}{2}$. The bending moment at the centre will be

$$M_e = \frac{pr}{2} \cdot r - \frac{pr}{2} \cdot \tfrac{2}{3}r = \frac{pr^2}{6}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (56)$$

But for a solid rectangular section we have

$$M_e = \tfrac{1}{6}fbh^2, \quad \text{or, for } b, = 1;$$

$$M_e = \frac{pr^2}{6} = \frac{fh^2}{6}, \quad \text{or} \quad f = \frac{pr^2}{h^2}; \quad . \quad . \quad . \quad . \quad . \quad (57)$$

whence

$$h = r\sqrt{\frac{p}{f}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (58)$$

where h = thickness of plate in inches;
r = radius of plate " " ;
f = stress in extreme fibre in pounds per square inch;
p = pressure on plate in " " " "

From a very elaborate analysis, Prof. Grashof finds for this case

$$h = r\sqrt{\frac{5p}{6f}} = 0.91r\sqrt{\frac{p}{f}}.$$

(*b*) *Square Flat Plate Supported at the Periphery and Uniformly Loaded.* —Since the corners are more distant from the centre and therefore carry a less proportion of the load, we may assume that the opposite sides act independently, so far as the bending moment at the centre is concerned. On this assumption the plate may be regarded as supported at two sides only and loaded with one-half the actual load, whence we have

$$M_e = \tfrac{1}{16}pbl^2 = \tfrac{1}{6}fbh^2, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (59)$$

or

$$h = l\sqrt{\frac{3}{8}\frac{p}{f}} = 0.61l\sqrt{\frac{p}{f}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (60)$$

where l = length of one side of the square plate.

(*c*) *Same Cases when the Plates are Fixed in Position at their Peripheries.*—Since the maximum bending moment on a beam fixed at the ends and uniformly loaded is only $\frac{2}{3}$ that of a beam supported at the ends and

* These proximate solutions are offered as illustrative of simple approximate methods which may often be applied to very complicated problems of this class.

similarly loaded, we may assume the same relations would hold here, thus giving for a circular plate, rigidly held,

$$f = \frac{3}{4}\frac{pr^2}{h^2}, \quad \text{or} \quad h = \frac{r}{2}\sqrt{\frac{3p}{f}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (61)$$

For a square plate, rigidly held,

$$f = \frac{9}{32}\frac{pl^2}{h^2}, \quad \text{or} \quad h = 0.53l\sqrt{\frac{p}{f}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (62)$$

(*d*) *For Elliptical and Rectangular Plates.*—Here the plate fails by cracking along its greater axis; and since the deflection of a beam for a given load is as the cube of the length, it is evident that the ends carry but a small part of the total load. Where the longer axis is more than twice the shorter one, we may neglect these end bearings entirely when we have the case of a flat plate supported at two opposite sides, which then becomes a simple beam: and this is the proper assumption to make in such a case. Making this assumption, and calling b the smaller dimension of the opening, we have

$$f = \frac{3}{4}\frac{pb^2}{h^2}; \quad \text{or} \quad h = \frac{b}{2}\sqrt{\frac{3p}{f}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (63)$$

Prof. Bach gives for this case

$$f = \frac{Ca^2b^2p}{(a^2 + b^2)h^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (64)$$

where C is somewhere between $\frac{2}{3}$ and 1.

When the longer axis is about $1\frac{1}{2}$ *times the shorter*, as is common with manhole-covers, assume that $\frac{3}{4}$ of the total load is carried at the sides, thus giving, from (64),

$$f = \frac{3}{4}\cdot\frac{3}{4}\frac{pb^2}{h^2}, \quad \text{or} \quad h = \frac{3b}{4}\sqrt{\frac{p}{f}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (65)$$

CHAPTER VI.

THE RESILIENCE OF MATERIALS.

51. Resilience Defined.—*Resilience* is literally the springing back of a deformed body after the deforming force has been removed. As used in mechanics, however, it is the work done by the body in this springing back, which is the same as the work done on the body in deforming it, so long as this is inside the elastic limits. Beyond the elastic limit the work of deformation always exceeds the work given back by the body. The body then does not fully recover its initial position, shape, or dimensions. Sometimes the work of deformation, whether inside or beyond the elastic limit, is spoken of as the resilience, but this is improper. *The resilience proper is the amount of work, or energy, in foot-pounds, which can be stored in an elastic body, up to a given stress per square inch, and which can be given out again by the body as useful work, if desired.** That portion of the energy spent in deforming the body but not given back as resilient work is spent in permanently deforming the body, by causing the particles to move or slide over each other, thus developing heat. The elastic deformation of a body does not develop heat. Since work is measured by a force acting over a distance, the work of deformation may be measured by the product of the deforming force into the distance through which it acts. But the deforming force is zero at first and increases uniformly as the deformation increases (inside the elastic limit); hence the total work done in deforming a body is the average value of the force into the total deformation. Since the force increases uniformly with the deformation, its average value is always one half its final value (inside the elastic limit), so that *the work of deformation, or the energy stored in the body, is one half the product of the final force (or resistance), into the deformation.* Inside the elastic limit the stress-diagram (for all kinds of stress) is a straight line, and here also the resilience, or work given back, is equal to the work of deformation. Hence the elastic resilience is equal to the triangular area of the stress-diagram, included between this curve, the axis of deformation, and an ordinate parallel to the axis of loads, to the extremity of the locus developed. As similar areas are to each other as the squares of their like

* This is the sense in which Young first used the term in 1807, but he did not so clearly define it since he assumed bodies to be perfectly elastic to rupture.

parts, or dimensions, it is evident *that the elastic resilience, or energy, stored in a body is as the square of the unit stress under the final load, this unit stress being equal or proportional to the load ordinate in the stress-diagram.* It will be shown hereafter that this is true for all kinds of resilience, both inside and beyond the elastic limit.

52. Three Varieties of Resilience.—There are three kinds of resilience commonly recognized, namely, of tension or compression, of bending, and of torsion. These, of course, correspond to the three corresponding kinds of stresses and deformations. It will be shown below that *the elastic resilience of a body, in foot-pounds or inch-pounds, is always equal to the product of three factors, namely:*

(*a*) *A numerical coefficient, which is different for each of the three kinds of resilience, and for different forms of cross-section, and for different methods of applying the external forces.*

(*b*) *The factor* $\frac{f^2}{E}$, *or the square of the maximum stress divided by the modulus of elasticity.*

(*c*) *The volume of the body.*

That is to say, for any particular kind of stress and form of cross-section the elastic resilience varies directly as the square of the stress-intensity and as the volume of the body, and inversely as its modulus of elasticity, or

$$R_s = k\frac{f^2}{E} \,.\, \text{volume.} \qquad \qquad (1)$$

That is to say, *the resilience, or energy, which can be absorbed, or stored, in a body of a given material and form, up to a given fibre-stress, is no function of the relative dimensions of the body, but only of its volume.* In other words, one cubic inch of steel will absorb and give out the same amount of work, or energy, as the same volume of fine wire, if the load is applied in the same manner, or if the stress is of the same kind, so long as the form of cross-section remains the same.*

53. Resilience a Measure of the Ability of a Body to Resist a Shock or Blow.—The magnitude or effect of a blow, or of a falling body, is measured by the energy stored in the moving body at the instant of impact. In the case of a body which has fallen freely in space under the action of gravity, its energy is Wh, where W is the weight of the body (force of gravity), and h is the distance through which the body has fallen freely (distance through which the force of gravity has acted). In any case, the energy of the body is $\frac{Wv^2}{2g}$, where v is the velocity in feet per second, and g is the acceleration

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* When the stress is direct (tension or compression), the form of cross-section is immaterial. In bending and torsion, however, the form of section is important.

of the force of gravity, or 32 feet per second. If a moving body, as a falling weight, is stopped by striking a fixed solid body, which is here assumed to be a test specimen, the energy of the moving body is spent in one or all of the following ways:

(*a*) In deforming the moving body itself, either within or beyond its elastic limit.

(*b*) In a local deformation of both bodies at the surface of contact, within or beyond the elastic limit.

(*c*) In moving the fixed body as a whole, with an accelerated velocity, the resistance consisting of the inertia of the body.

(*d*) In moving the fixed body against its external supports and resistances.

(*e*) Finally, in deforming the fixed body as a whole against the resisting stresses developed thereby.

If the moving body be very hard and rigid; if the surfaces of contact are comparatively unyielding; if the specimen have a small mass as compared to the moving body, and if it be very rigidly supported upon or against a very great mass or weight which is relatively unyielding; and, finally, if the specimen which is to receive and absorb the energy of the blow is quite yielding or flexible, and in short if there is nearly absolute rigidity in all parts of the apparatus except in the body struck, and if this yields only as a whole and not at the point of contact or at its supports,—then, and only then, can nearly all the energy of the moving or falling body be absorbed by the deflection or stretch or compression or twisting of the specimen. It is practicable, by making the energy of the falling body consist mostly of weight and only to a small degree of velocity, that is, by having a heavy weight drop through a short distance, to absorb upwards of 90% of it in the specimen. It goes without saying that it is impossible to get it all stored in the specimen under any circumstances; and if great care is not exercised in arranging the test, but a very small percentage may be given over to the specimen, the rest being dissipated in the other ways named above.

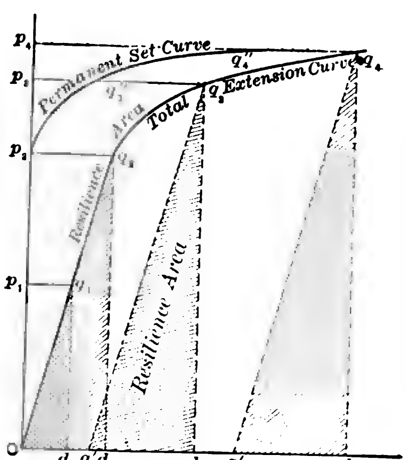

FIG. 51.

In the stress-diagram shown in Fig. 51 let the vertical ordinate represent total resistance in pounds and the horizontal ordinate represent deformation of the body, as a whole, measured at the point of contact, in inches; whether this deformation be a bending, extension, compression, or twist is not now material. When this body has been deformed to d_1 it is resisting this action with a force of p_1; when deformed

to d_2 it is resisting with a force of p_2, etc. When the deformation passes the elastic limit the resistance does not increase as rapidly as the deformation, and hence the diagram is no longer a straight line, but becomes curved. A deformation of d_3 now develops a resistance of p_3, and d_4 of p_4, etc.

Now since the *work of resistance* is the sum of the products of the instantaneous resistances into the corresponding deformations, it is properly represented by the area of the stress diagram up to the maximum deformation and resistance. That is to say, the work done on the body to deflect or deform it to d_1 is indicated by the area of oq_1d_1; the work required to deform it to d_2, and also the energy stored in the body when deformed to this point, is indicated by the area oq_2d_2, etc. So long as the point q falls on the stress diagram inside the elastic limit, this amount of energy stored will all be given back again by the body. But when this point q falls beyond the elastic limit point of the diagram, the body is no longer able to fully recover its original form, but it remains permanently deformed. The amount of this permanent set can always be found by dropping lines $q_3q_3'q_4q_4'$, etc., from the extremity of the diagram which marks the maximum load imposed, *parallel to the straight portion of the curve.* These lines are the *return paths* which the specimen follows on the removal of the deforming forces or loads. They are always parallel to the elastic path of the body, or to that part of the curve below the elastic limit. This is true for all kinds of stresses and diagrams, whether tension, compression, bending, or torsion; and whether the vertical ordinate represents total loads or resistances, or loads per square inch, or intensities of stress on extreme fibres.

In case the specimen has had to absorb an amount of energy, or work, represented by the area oq_3d_3, therefore, it will give back only so much as is represented by the area $q_3'q_3d_3$. The remainder, oq_3q_3', represents work which has been spent in permanently deforming the specimen, and which it can never give back, this having been transformed into heat by friction. Under our definition of resilience, therefore, we should have to say that the resilience of the specimen, for the resistance p_3, is $q_3'q_3d_3$, and not the full area oq_3d_3. This latter represents the work done in deforming the specimen, but it cannot properly be called *resilience.* Similarly, when the body is distorted to d_4 with a developed resistance of p_4; the resilience now is $q_4'q_4d_4$, and oq_4q_4' has been lost in the permanent deformation of the specimen, or in heat.

The student will readily perceive that the areas of the triangles whose bases are od_1, od_2, $q_3'd_3$, and $q_4'd_4$, respectively, are to each other as the squares of these bases, or as the squares of their altitudes, p_1, p_2, p_3, and p_4, respectively, since they are all similar, their sides being parallel. If their altitudes represented stresses per square inch, which they might, then we could say the resilience of this specimen varied as the square of the stress developed in it, as stated in Art. 52, and as will be further shown analytically, whether this maximum stress be inside or beyond the elastic limit.

Thus far in studying Fig. 51 we have spoken of the "work of deformation" without stating whether this work was developed by a load slowly applied, or by one quickly applied, as by a falling weight. In fact it does not matter how this work is done, a given number of foot-pounds of energy producing exactly the same effect, and developing the same stress diagram, provided we assume that *all* the energy of the quickly applied load goes into the specimen, to produce this particular deformation. This conclusion is also based on another *assumption*, which is, *that the relation between the deformation and its corresponding resistance developed in the body is the same for a deformation produced suddenly as for one produced by a slower application of external force.* This equality of relationship has never been shown, as between static and impact applications of the load; but it is probable that this relation is very nearly independent of time, inside the elastic limit, and with brittle bodies up to rupture, since it is in this case a molecular resistance to relative deformation, and not a resistance to flow or relative displacement. In the case of plastic or ductile bodies, however, it has been shown that beyond the elastic limit the stress diagrams developed by impact and by static loads are very different, the former being in the case of soft iron wire some 30% greater in area. This means that for such materials the actual energy absorbed by the specimen under impact is some 30% more than it is under a static load. See Fig. 52.

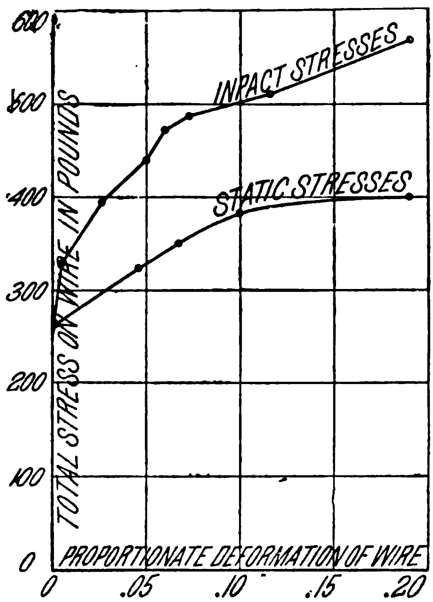

FIG. 52.—Showing that a greater impact stress is required to produce a given deformation. (*Fr. Com. Rep.*, vol. II. p. 344.)

Assuming now that all the energy of a blow is spent in deforming the specimen in the manner represented on the static stress-diagram, we come to this very important conclusion: *the energy of the blow, in foot-pounds, is equal to the area of the stress-diagram developed by that blow, properly evaluated to the scales of the drawing, when this diagram is drawn to co-ordinates representing deformation and resistance thereto.* Thus if a weight falls on a body, as a beam, and *if we may assume* that very little of the energy spends itself otherwise than in bending the specimen, if the specimen deflects by the amount d_2, for instance, then we assume that the corresponding resistance at the instant of maximum deflection is p_2, and that the energy of the blow was somewhat greater than the area oq_2d_2; or by know-

ing the energy of the falling body (Wh), and observing the deflection produced, we could determine the amount of energy absorbed by the specimen *if we only had a stress-diagram of this specimen under impact, as shown in Fig.* 52. Few such diagrams have ever been obtained. It has been customary to use for this purpose static test-diagrams, carried beyond the elastic limit, and perhaps to failure. This, of course, destroys the specimen for impact tests; but by having two specimens, presumably just alike, a static test may be made on one of them, from which a static stress-diagram can be drawn, and then the impact test on the other specimen can be interpreted by this static diagram. Having done this, we should still fail to find the area of stress-diagram developed by a single blow fully equal to the energy of the blow, because of the dissipation of a portion of this energy in other ways, and also because the impact stress-diagram lies above, or outside of, the static diagram.

It is common to test materials by means of falling weights, and often the height of drop is regularly increased until failure occurs. Let us follow the course of such a test, referring again to Fig. 51. Thus we will suppose all the energy of the blow goes into the specimen (or it would serve as well to suppose a certain fixed percentage is absorbed by the specimen), and that the first blow deformed the specimen to d_1, the second to d_2, the third to d_3, and the fourth to d_4. Now what were the energies of these blows, if all went into the specimen each time? Evidently the energy of the first blow was that indicated by the area oq_1d_1; of the second by oq_2d_2; of the third by oq_3d_3; and of the fourth by $q_3'q_3q_4d_4$. Thus we see that all the first area is included in the second, all of both first and second in the third, and a large part of the third ($q_3'q_3d_3$) in the fourth. These areas are therefore not mutually exclusive, so that the sum of the energies of all the blows (ΣWh) is not equal to the total area of the stress-diagram developed by them, oq_4d_4. Neither is the energy of the last blow equal to this area, and in fact there is no relation between the total area of the stress-diagram, oq_4d_4, and the energy of one or all of the blows given. If we now add to this statement the evident fact that we can never know in practice what portion of the energy of any blow is spent in deforming the specimen (and often we cannot tell what this proportion is within over 50%, and sometimes it has been assumed that it all went into the specimen when there could not have been more than *five per cent* of it so spent !), it becomes patent that NO ABSOLUTE CONCLUSION WHATEVER CAN BE BASED ON IMPACT TESTS. Some relative conclusions may be drawn by subjecting two or more like specimens to exactly identical treatment and finding which withstands the greater number of blows. Even then *the apparent relative strength depends largely on what particular magnitude of blow be selected for making tests.* Also a very small difference (apparently) in the character of the foundation on which the specimen rests may make a very great difference in the *percentage* of the total energy which goes into the specimen. Hence such

comparative tests should always be made on the same foundation, and all the elements of the test exactly duplicated.*

In order to obtain the absolute characteristics of any material, a complete stress-diagram should be obtained by static tests. The area of such a diagram up to its elastic limit indicates the total energy of the single shock or blow it could withstand up to that limit, or without taking a permanent set (provided this energy all went into the specimen), and the area of this diagram, up to rupture, indicates the total energy of the single blow it could absorb without actually breaking.

It must also be observed that in addition to the impact stress being greater for given deformations, beyond the elastic limit with ductile materials, the total elongation at rupture is also much greater when produced by a sudden blow than when produced in a static test, and hence the area of the stress diagram thus developed may be very much larger than the static-test diagram on the same material.

For an absolute measure of a given material to withstand a shock or blow, therefore, it is necessary to give it a static test in some kind of a testing-machine, whether this test be in tension, or in compression, or in cross-bending, or in torsion. Then *the area of the stress-diagram up to the elastic limit, divided by the volume of the specimen under test, is a measure of the ability of the material, per unit of volume, to absorb and give out energy, or to resist repeated shocks without injury; and the total area of the stress-diagram is its measure to resist a single blow without rupture.*†

It is necessary in this connection to guard the student against several misconceptions.

(*a*) By a "slow" or "static" test is meant such a gradual imposition of the load as will give to the moving parts an inappreciable velocity, or momentum, or *vis viva.* Evidently any ordinary test in a testing-machine fulfils this condition.

(*b*) By an impact test, or a shock, or a blow, is meant a genuine striking or impact, in which the force of the blow is nearly all due to the speed or velocity of the moving body or falling weight, and only slightly due to its static weight alone.

(*c*) Aside from the two methods which alone have been under discussion in this article, there is another method of loading, called a "sudden imposition of load." Thus in the case of placing a load on a beam, if the load be brought into contact with the beam, but its weight sustained by external means, as by a cord, and then this external support be

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*It is not uncommon to find impact tests described by giving only the weight of hammer and height of fall, with no description of the character of the supports. It has also been customary to rest stamp-mills on spring-timbers to lessen the force of the blow!

† Except that for ductile materials, in which the impact stress-diagram is greater than the static stress-diagram, as shown in Fig. 52.

suddenly (instantaneously) removed, as by quickly cutting the cord, then, although the load is already touching the beam (and hence there is no real impact), yet the beam is at first offering no resistance, as it has as yet suffered no deformation. Furthermore, as the beam deflects the resistance increases, but does not come to be equal to the load until it has attained its normal deflection. In the meantime there has been an unbalanced force of gravity acting, of a constantly diminishing amount, equal at first to the entire load, but now reduced to zero when the resistance has come to be equal to the load, at the normal deflection. But at this instant both the load and the beam are in motion, the hitherto unbalanced force having produced an accelerated velocity, and this velocity of the weight and beam gives to them an energy, or *vis viva,* which must now spend itself in overcoming an *excess* of resistance over and above the imposed load, and the whole mass will not stop until the deflection (as well as the resistance) has come to be equal to *twice* that corresponding to the static load imposed. Henco we say the effect of a suddenly imposed load is to produce twice the deflection and stress of the same load statically placed. It must be evident, however, that this case has nothing in common with either the ordinary "static" tests of structural materials in testing-machines, or with impact tests. It is introduced here to prevent a confusion of mind in these matters often found to exist with persons whose conceptions of such problems in mechanics are not clear.

54. Resilience Areas in Stress-diagrams.—It was shown in the previous article, in discussing Fig. 51, that the shaded triangular areas represented the resilience of the specimen for the several loads imposed. It will now be shown that these areas may be represented as one figure with continuously added increments.

Referring again to Fig. 51, if the permanent set, oq_3, be laid off on p_3q_3 from p_3 giving q_3'', oq_4 on p_4q_4 from p_4 giving q_4'', etc., and drawing a curve through these points from p_2, the elastic-limit stress, the curve so drawn may be called the *curve of permanent sets.* If we now regard the space intercepted between this and the stress-diagram, it is evident that the length of the horizontal intercept increases directly as its altitude above the horizontal axis, since these intercepts are the bases of the similar triangles on the horizontal axis, whose apexes lie in those horizontal lines. This curved area $op_2q_3''q_4''\ldots q_4q_3q_2o$ is therefore a more general type of a true triangle, whose area is simply equal to its upper base (the horizontal intercept which equals the elastic deformation) into one half its altitude (which is the maximum stress produced, or load imposed). In other words, the following triangles are equal, because they have equal altitudes and bases. Since they also have equal angles at the vertex, they are, in a more general sense, similar triangles:

$$\text{Triangle } oq_2d_2 = \text{triangle } op_2q_2;$$
$$\text{Triangle } q_3'q_3d_3 = \text{triangle } op_3q_3''q_3q_2o;$$

Triangle $q_4'q_4d_4$ = triangle $op_4q_4''q_4''q_4q_3q_2o$.
etc. etc.

Hence by simply constructing both the stress-deformation and the stress-set curves, we may indicate directly the resilience or work which the specimen will be able to give back after having been stressed to any assigned limit.* The value of this resilience is always one half the product of the final stress into the difference between the final distortion and the permanent set; or, *in general, whether inside or beyond the elastic limit, the resilience is equal to one half the product of the final load or stress into the elastic deformation.*†

55. Resilience of Bodies under Direct Stress.—When a body of a uniform cross-section and of a definite length is subjected to the action of external forces, producing direct tension or compression, the deformation produced in the body, from eq. (2), Chapter I, is $\alpha = \frac{pl}{E}$. If A = the cross-section of the body, then the total external force applied is $P = pA$. The total external work is then

$$\frac{P\alpha}{2} = \frac{pA}{2} \cdot \frac{pl}{E} = \frac{1}{2}\frac{p^2}{E}Al. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

But since this is equal to the internal work of resistance, and since Al = volume of the specimen, we have

$$R_d = \textit{resilience in direct stress} = \frac{1}{2} \cdot \frac{p^2}{E} \cdot \text{volume}; \quad . \quad . \quad . \quad (3)$$

or per unit of volume,

$$r_d = \frac{1}{2}\frac{p^2}{E}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Since p and E are given in pounds per square inch, the volume must also be in cubic inches.

If p is made equal to the elastic limit of the material, the corresponding value of r_d is the *primitive elastic resilience in inch-pounds per cubic inch.* Beyond the elastic limit, the elastic resilience is indicated by the triangles $q_2'q_2d_2$, $q_4'q_4d_4$, etc., in Fig. 51, corresponding in each case to the new or artificially-raised elastic limits p_2, p_4, etc. These subsequent elastic resilience values may be called the *artificially-raised elastic resilience.*

As defined in the previous article, all resilience is elastic resilience, but the term "elastic" is retained here in order to insure that it is not confused with the term "total resilience," which is sometimes misused and made to mean the total area of the stress-diagram, which the author of this work will not admit is resilience in any sense.

56. Resilience in Cross-bending.—The deflections of beams loaded and supported in different ways, in terms of the stress on the extreme fibre, are

* When the stress passes a maximum and both these curves begin to descend, the included area here becomes negative.

† True under the author's definition of resilience, but not true when this term is made to mean the work or energy *absorbed* instead of the energy *given back.*

given in column four of the table on pages 62–65. For any case the deflection may be represented by the term $k\frac{fl^2}{Eh}$, where k is a numerical coefficient which varies for the different cases, but the values of which are given in that table. Since the resilience of a beam when developed by falling weights, or other impact loads, would produce deflections corresponding to concentrated loads, only concentrated-load deflections as given in the table need be here considered. Thus, for a beam supported at the ends and loaded at the centre, the deflection, in terms of the stress on the outer fibre, is $\Delta = \frac{1}{6}\frac{fl^2}{Eh}$. But the load which will produce the stress f on the outer fibre is (see column five of table) $P = \frac{8fI}{lh}$. The external work done on the beam in deflecting it, which must equal the internal work of resistance, or the resilience, if f is inside the elastic limit, is

$$\text{Resilience} = \frac{P\Delta}{2} = \frac{2}{3}\cdot\frac{f^2}{E}\cdot\frac{lI}{h^2}. \quad \ldots \ldots \quad (5)$$

For a solid rectangular cross-section, $I = \frac{1}{12}bh^3$.
Substituting this in eq. (5), we have

Resilience of a solid rectangular beam loaded at the centre and

$$\textit{supported at the ends} = \frac{1}{18}\cdot\frac{f^2}{E}\cdot bhl = \frac{1}{18}\cdot\frac{f^2}{E}\cdot \textit{volume}. \quad . \; . \quad (6)$$

If the bending moment had been uniform throughout its length, as is the case with a spiral or helical spring when under a bending stress, the movement of one end of the spring should be determined, and this multiplied by one half the final force applied at this point. But since the internal work of resistance is always equal to the external work of deformation, we may measure up the internal work and call this the resilience. The case of a beam (or a spring) under a uniform bending moment is a favorable case for this purpose. Thus assume a spiral or helical spring made of a steel bar having a rectangular cross-section whose original dimensions were l, b, and h. When coiled into a spring (the dimensions of the coil being immaterial for our purpose) and a couple producing bending moment applied to it, thus developing in the spring throughout its entire length a moment of resistance which we will suppose is such as to give rise to the elastic-limit stress f on the outer fibres throughout the entire length of the coiled bar, we are to measure up the total internal work of resistance, or the energy thus stored in the spring. Since the fibre-stress is uniformly vary-

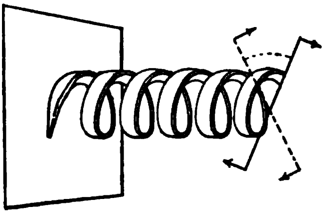


FIG. 53.

ing across the section of the bar, and is f on the outer fibres on each side, it is evident that

$$\textit{The stress on any fibre} = p = ay = f\frac{y}{y_1} = \frac{2f}{h}y, \quad . \quad . \quad . \quad (7)$$

where y = distance of fibre from neutral axis, and

y_1 = distance of outer fibre from neutral axis $= \frac{h}{2}$;

f = stress per square inch on outer fibre.

But

$$\textit{The stretch of any fibre} = \alpha = \frac{pl}{E} = \frac{2fl}{Eh}y, \quad . \quad . \quad . \quad . \quad (8)$$

where p = stress per square inch;

l = length of bar of which spring is composed;

E = modulus of elasticity.

Therefore

$$\textit{The work of resistance of any fibre} = \frac{2f^2}{E} \cdot \frac{l}{h^2} \cdot y^2. \quad . \quad . \quad (9)$$

The work of resistance of any zone of fibres bdy in area of cross-section, distant y from the neutral axis, would be $\frac{2f^2}{E} \cdot \frac{bl}{h^2} \cdot y^2dy$, and

$$\text{The total work of resistance} = \text{resilience} = R = \int_{-\frac{h}{2}}^{+\frac{h}{2}} \frac{2f}{E} \cdot \frac{bl}{h} \cdot y^2dy$$

$$= \frac{2f^2}{E} \cdot \frac{bl}{h^2} \int_{-\frac{h}{2}}^{+\frac{h}{2}} y^2dy = \frac{2f^2}{E} \cdot \frac{bl}{h^2}\left[\frac{y^3}{3}\right]_{-\frac{h}{2}}^{+\frac{h}{2}} = \frac{2f^2}{E} \cdot \frac{bl}{h^2} \cdot \frac{h^3}{12} = \frac{1}{6} \cdot \frac{f^2}{E} \cdot blh$$

$$= \frac{1}{6} \cdot \frac{f^2}{E} \cdot \text{volume of spring} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Comparing this with eq. (5) we see that fifty per cent more energy can be absorbed by a beam or spring when subjected to a uniform bending moment than when the moment increases uniformly from the ends to the centre, or from one end to the other,

57. **Resilience in Torsion.**—Referring to Fig. 23 we see that the external work is $\frac{Pa\theta}{2}$, where θ is the distortion angle and a = length of arm of the couple, whose forces are P. But from eq. (14), Chapter III, when the moment Pa is on the specimen the stress on the outer fibre is

$$f_s = \frac{2Pa}{\pi r^3}, \quad \text{or} \quad Pa = \frac{f_s \pi r^3}{2}.$$

Also, from eq. (15),

$$\theta = \frac{2Pal}{\pi r^4 E_s} = \frac{5Pal}{\pi r^4 E}, \quad \text{since } E_s = \tfrac{2}{5}E.$$

Substituting here the value of Pa above, we have

$$\theta = \frac{5 f_s \pi r^3 l}{2\pi r^4 E} = \frac{5}{2}\frac{f_s l}{rE}.$$

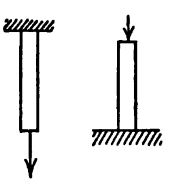
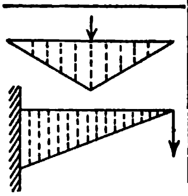
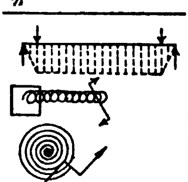
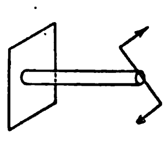
Combining this with the value of Pa again to get the value of $\frac{Pa\theta}{2}$, we have

$$\textit{Work of torsion on solid cylinder} = \frac{Pa\theta}{2} = \frac{5}{8}\cdot\frac{f_s^2}{E}\cdot\pi r^2 l$$

$$= \frac{5}{8}\cdot\frac{f^2}{E}\cdot \text{volume.} \quad . \quad . \quad . \quad . \qquad (11)$$

58. Comparative Resilience of Bodies under Different Kind of Stress.— For bodies of uniform cross-section we have the following table of values of resilience in *inch-pounds per cubic inch*, and their relative capacities to absorb and give out energy, taking the capacity in direct stress as unity.

COMPARATIVE RESILIENCE OF BODIES.

Figure.	Kinds of Stress.	Resilience in inch-pounds per cubic inch. $= r$.	Relative Capacity for Absorbing and Giving out Energy.
	Direct tension or compression.	$\frac{1}{2}\cdot\frac{f^2}{E}$	1
	Cross-bending with bending moment uniformly increasing longitudinally.	$\frac{1}{18}\cdot\frac{f^2}{E}$	$\frac{1}{9}$
	Cross-bending with bending moment uniform longitudinally.	$\frac{1}{6}\cdot\frac{f^2}{E}$	$\frac{1}{3}$
	Torsion.	$\frac{5}{8}\cdot\frac{f^2}{E}$	$\frac{5}{4}$

EXAMPLES ON PART I.

1. A section of a steel bar 1 in. in diameter and 8 in. long elongates 0.01 in. for an increase in tensile stress of 30,000 lbs. What is the modulus of elasticity?

2. What reduction in temperature would bring a wrought-iron bar, immovably fixed at its ends, to its elastic limit of 26,000 lbs. tensile stress per square inch? Take $E = 28,000,000$ and the coefficient of expansion $= 0.0000065$ per degree F. *Ans.* 142°.8

3. Find the proportionate change in *volume* of a brass cube which is subjected to a compressive stress in one direction of 10,000 lbs. per square inch. Take $E = 15,000,000$. What is its change in volume for a fluid pressure of this amount in all directions? *Ans.* 0.00023; 0.00069.

4. What is the shearing modulus of elasticity for steel if $E = 29,000,000$ and Poisson's ratio $= 0.27$? *Ans.* $E_s = 0.39E$.

5. Find the modulus of elasticity of steel from Fig. 6, making allowance for the locus cutting the vertical axis at 1000 pounds above the origin. Use the deformation of 0.001 and its corresponding stress-increment in pounds per square inch. From the same diagram find the elastic limit, the ultimate strength, and the percentage of elongation.

6. The following is a record of a test on cast iron:

Loads per square inch in pounds..........	1000	5000	10000	15000	20000	25000	30000	31040
Proportionate deformations..............	0	.00022	.00055	.00097	.00150	.00220	.00368	broke

Plot this record and determine from it: (1) The modulus of elasticity; (2) The apparent elastic limit; (3) The total percentage of elongation (by extending the plotted curve till the breaking load is reached); (4) The work required to break the specimen in foot-pounds per cubic inch of metal (obtained by finding the area of the diagram and evaluating it to the scales of the drawing, see Art. 53).

7. Assume a brick to be 8 in. long, 4 in. wide, and 2 in. thick. From equation (12), p. 31, find the relative crushing strength of the brick per unit area when tested flatwise, edgewise, and endwise, taking the strength of a cubical specimen of the same material as unity. *Ans.* 1.22; 0.89; and 0.83.

8. A stone cube two inches on a side has its edges chamfered or rounded so that the bearing surfaces are but 1.8 in. square. What is its total crushing strength as compared to the strength of a full cube? What is its strength per square inch of bearing-surface as compared to the strength per square inch of a full cube? (See Fig. 18.) *Ans.* 85 per cent; 103 per cent.

9. By how much is a centrally loaded column 12 in. square weakened by adding four inches of the same material to one side of the column without shifting the load? *Ans.* The maximum stress in the column is increased by 31¼ per cent.

10. A steel rod 1/4 in. in diameter and 30 in. long is used as a torsional spring for closing a door. What will be the increase in the moment of torsion from giving the rod an additional twist through 90°, the shearing modulus of elasticity being taken as 12,000,000? What will be the maximum increased shearing stress in the rod due to this angular movement? What will be the increase in the force required to hold the door in this position, the door-knob being 30 inches from the hinges? *Ans.* 244 inch-pounds; 78,500 lbs. per square inch; 8 pounds.

11. A wooden beam 8 in. by 16 in. in cross-section and 20 ft. long carries a uniform load of 1000 lbs. per running foot. Find the maximum direct stress on the outer fibres and the maximum shearing stress in the beam.
Ans. { 1170 lbs. per square inch direct stress;
117 " " " " shearing "

12. For the same beam and load as in Ex. 11, find the deflection of the beam, taking $E = 1,200,000$. If the deflection were observed to be 3/4 in., what would be the modulus of elasticity? *Ans.* 1.1 in deflection; 1,760,000 modulus.

13. A flitched beam is composed of two sticks 4 in. by 16 in. by 16 ft. long, and a steel plate 3/4 in. by 16 in. of the same length, and carries a load of 2000 pounds

per running foot. Find the portion of the load carried by each part, the maximum fibre-stresses resulting, and the deflection at the centre, taking $E = 30,000,000$ for the steel and 1.500,000 for the timber

Ans. { Steel: 1305 lbs. per foot; 15,660 lbs. ; 0.25 inch.
Timber: 695 " " " 782 " " "

14. How many foot-pounds of energy per pound of steel can be stored in a steel helical or spiral spring coiled about an axle, by winding it up until the stress in the outer fibre is 80,000 lbs. per square inch, E being taken equal to 30,000,000 ? *Ans.* 17.8.

15. How much would such a spring weigh which could absorb the energy of a street-car weighing 20,000 lbs., and moving at the rate of six miles per hour on a down grade just sufficient to overcome the frictional resistances? Would the size of the cross-section of such a spring affect its necessary weight?

Could such a spring be designed so as to reach this fibre-stress when the car had stopped, and also so as to be exerting the maximum torsional moment on the car-axle without causing the wheels to slip? Is such a device practicable?* *Ans.* 2315 lbs.

16. To what extent can energy be stored in metallic springs of any sort? Could they ever be used for the storing of motive power? (This has often been attempted.)

17. A pendulum, mounted on knife-edges, weighs 50 lbs., and its centre of gravity is 8 feet from the pivot-supports. It is moved to an angle of 30° from the vertical, and is allowed to swing and strike the centre of a cast-iron bar 1 in. square, resting on absolutely rigid supports (or assumed to be such) 2 feet apart. The pendulum in falling breaks the bar and moves a horizontal distance of 24 in. beyond its true vertical position before it comes to a stop. What is the shock-resisting capacity of the iron in inch-pounds per cubic inch of metal in cross-breaking under a concentrated load? *Ans.* 20.4.

18. Assuming the stress-diagram of a static test of such a bar in cross-bending to be a triangle, what would its final deflection be if the breaking load were such as to correspond to a modulus of rupture of 40,000 lbs. per square inch? *Ans.* 0.88 inch.

* This is a favorite device with "car-starter" inventors. See an article by the Author in *Jour. Assc. Eng. Socs.*, vol. IV. p. 293.

PART II.

MANUFACTURE AND GENERAL PROPERTIES OF THE MATERIALS OF CONSTRUCTION.

CHAPTER VII.

CAST IRON.

GENERAL CLASSIFICATION OF IRON AND STEEL.

59. Importance of the Subject.—While the use of iron in a small way, for offensive and defensive weapons of war and for utensils, is doubtless older than authentic history,* it is only since its manufacture has become possible on a grand scale, by the aid of steam-power, that it has become a common material of engineering and architectural construction. It has now nearly replaced the use of timber in engineering works, and it is rapidly replacing the use of wood, stone, and brick in architectural designing. So dependent now are all kinds of construction on the use of iron, that the condition of the iron-manufacturing industry is universally regarded as a true index of the general state of trade and commerce the world over. Since iron, therefore, in its various states, is more used in engineering construction than all other kinds of materials combined, a corresponding amount of space is given to a study of it in this work.†

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* There is now in the British Museum (*a*) a sickle-blade found by Belzoni under the base of a sphinx near Thebes; (*b*) a blade found by Col. Vyse embedded in the mortar of one of the Pyramids; (*c*) a portion of a cross-cut saw exhumed by Layard at Nimroud. These may be of meteoric origin. The reason more specimens of iron and steel are not found may be due, however, to their rapid oxidation when exposed to air and moisture. The stone and bronze implements have resisted this action, and hence many have assumed that in the "stone" and "bronze" ages no iron was in use.

† A chronological review of the greatest discoveries and inventions in iron manufacture is here given:

4000 B.C. to about 1500 A.D. } Wrought iron by the direct process from the ore in small quantities by means of charcoal, and this made into cement- or blister-steel.

About 1500 A.D.—Cast iron made in Germany with charcoal.

1620–1735. *Cast-iron* made by *Dud Dudley* in England with coke, but the prac-

60. Classifications of Iron and Steel.—Iron and steel may be classified according to its qualities, structure, and composition, or according to its methods of manufacture. Apparently the former is the more significant basis of classification, but in English- and French-speaking countries the latter basis has come to be universally adopted. We will, however, here first classify these products according to their more significant qualities (the method used in Germany).

IRON AND STEEL CLASSIFIED ACCORDING TO QUALITIES.

- Malleable.
  - Cast, when molten, into a malleable mass or ingot.
    - *Ingot Iron*—cannot be hardened by sudden cooling.
    - *Ingot Steel*—can be hardened by sudden cooling.
  - Aggregated from pasty particles without subsequent fusion (puddling process).

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tice lapsed till revived in 1735 by *Abraham Darby*. Blast from leather bellows driven by water-power.

1740. Cement-steel *melted in crucibles* by *Huntsman* near Sheffield, England.

1760. The *steam-engine* of *Watt* applied to produce the blast for making pig iron, and to drive rolls and hammers in working the wrought iron and steel.

1783–4. *Grooved rolls* of various forms, driven by the steam-engine, and *wrought iron* made from pig iron by "dry-puddling," both by *Cort*, England. White iron used in the "dry" process. These inventions lie at the base of the supremacy of Great Britain in the iron trades.

1829. *Hot blast*, used in blast-furnaces in Scotland by *Neilson*, thus greatly cheapening the cost of production.

1830. The "*wet-puddling*" process of making wrought-iron, or "pig-boiling," introduced by *J. Hall*, England.

1840. Use of *manganese* in making crucible cast steel, introduced at Sheffield by *J. M. Heath*, which reduced the cost of steel by 50 per cent.

1856. The *Bessemer process* of making steel, patented by *Sir Henry Bessemer* in England (son of a French refugee, born 1813), this "being of far more importance to the world than all the gold of California and Australia."

1861. Invention of the *regenerative gas furnace* by *Sir W. Siemens* in England (born in Hanover, 1823), and educated at the Magdeburg Polytechnicum and at Göttingen).

1868. Application of the *Siemens* furnace to the *open-hearth* process of making steel by *P. and E. Martin* in France, thus originating the *Siemens-Martin* process of steel-making now employed for nearly all the soft and mild steel used in structural work, and for steel castings.

1878. The invention of the *basic process* of making steel, by which the phosphorus of the ore is eliminated, by *S. G. Thomas* and *P. C. Gilchrist* (cousins) in England. (First public demonstration April 4, 1879.) By this process the range of ores which can be used for steel-making is enormously increased, especially in Europe, while it is used often in America.

Weld Iron—cannot be hardened by sudden cooling.

Weld Steel—can be hardened by sudden cooling.

Semi-malleable.

Steel Castings—malleable metal cast into final forms.

Malleable Cast Iron—non-malleable metal (cast iron) cast into final forms and then brought to a semi-malleable condition.

Non-malleable.

Cast Iron.

Hard Cast Steel.

The significant criterion here employed to distinguish between iron and steel consists in the hardening effects of sudden cooling from a bright-red heat. This is not a very satisfactory criterion, however, since all such metal is hardened somewhat by sudden cooling.

What are commonly known as wrought iron and steel, however, are made by radically different processes—one being the formation of the product in a melted, or liquid, state and then casting it into a mould, forming what is called an ingot; the other consisting in forming the product in a pasty or spongy state in a bath of melted or liquid foreign matter, from which it is lifted and immediately forged or rolled. When formed in the melted state it is purified from all foreign matter except such as enters into its own composition, while when formed in the pasty or spongy state, in a bath of melted foreign matter, a considerable proportion of this foreign matter or slag is, of necessity, lifted out with the pasty aggregation, called a "puddle-ball," and some of this slag remains distributed through the iron even after it is rolled, thus giving it a kind of fibre or grain. While, therefore, the mechanical qualities of the puddled product may be almost identical with those of the cast product, there is always a sufficient difference in their structure, resulting from the radical differences in their methods of manufacture, to clearly distinguish them by simply examining the fracture, and to warrant a classification on this basis also: and this is the customary basis of classification in this country.* We have, therefore,

IRON AND STEEL CLASSIFIED ACCORDING TO METHOD OF MANUFACTURE.

Malleable.

Wrought Iron—rolled or forged from a puddle-ball; it contains slag and other impurities, and cannot be hardened by sudden cooling.

Steel—rolled or forged from a cast ingot and free from slag and similar matter.

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* It has also been recommended by a sub-committee of the recent French Commission.

Soft Steel—will weld (with care), and cannot be hardened by sudden cooling (Ingot Iron). Same uses as Wrought Iron.
Medium Steel—will weld imperfectly except by electricity), and will not harden by sudden cooling. Used in Structural Work.
Hard Steel—will not weld, and will harden by sudden cooling. Tool-steel, Spring-steel, etc.

Semi-malleable.
Steel Castings—Malleable metal cast into final forms.
Malleable Cast Iron—non-malleable metal cast into final forms and then brought to a semi-malleable condition.

Non-malleable.
Cast Iron.
Hard Cast Steel.

Neither of these classifications must be construed too rigidly, but they fairly define the common usage, so far as the employment of these materials in engineering design is concerned.

THE PHYSICAL PROPERTIES OF CAST IRON.

61. General View.—While cast iron has been known and commonly employed since the Middle Ages, it has not been critically and scientifically studied till within a very few years. In the last quarter of a century, the attention of metallurgists engaged in iron-manufacturing industries has been almost wholly confined to the manufacture of steel. The great advances which have been made in this direction have caused cast iron to be very largely replaced by steel in structural designing, and in other directions, and since 1885 steel has also been cast in final forms, the same as cast iron, so that the use of cast iron has been very much diminished, relatively to the total iron and steel output. For many purposes, however, cast iron will probably never be replaced by any other material, especially since great improvements have been made in this direction, as a result of scientific study and experiments devoted in recent years to the manufacture of cast iron.

Much of the matter here given on this subject has been quoted directly from the *Metallurgy of Iron* by Thomas Turner, Associate of the Royal School of Mines, England. This work was published in 1895 and contains the latest results of scientific research on the subject there treated.*

62. General Properties.—"Cast iron consists of metallic iron, together with at least 1.5 per cent of carbon, It also contains silicon, sulphur, phosphorus, manganese, and other elements in greater or less proportion, but these may be regarded as impurities, though their presence is often useful or even necessary for the purposes for which cast iron is applied. The pro-

* When not otherwise credited the quoted paragraphs are from this work. (Chas. Griffin & Co., London, and Lippincott, Philadelphia.)

portion of elements other than iron is usually about 7 per cent of the total weight, though this varies considerably and is sometimes very much more. Cast iron is fusible at a temperature of about 1200° C. (2200° F.); when cold it is hard and brittle, some varieties being much more so than others; it is not malleable or ductile, like wrought iron or mild steel, nor can it be hardened and tempered like ordinary carbon steel. The iron-founder distinguishes between *pig iron*, or the form in which the metal is obtained from the blast-furnace, and *cast iron*, or the form it assumes after it has been again melted; but no such difference is recognized by the chemist, and pig iron is merely a variety of cast iron which is produced in a particular form."

63. Carbon in Cast Iron.—" Cast iron, when fused, consists of a saturated, or nearly saturated, solution of carbon in iron. The amount of carbon which molten iron can thus dissolve is about 3½ per cent of its own weight, though the solubility is largely influenced by the presence of other elements. With much chromium the maximum solubility of about 12 per cent of carbon is reached; with much manganese up to 7 per cent of carbon may be dissolved; while with about 20 per cent of silicon the minimum solubility of carbon is obtained, and only about 1 per cent of carbon then dissolves. Apart from special alloys, such as those mentioned, it is very unusual to meet with less than 2 per cent or more than 4.5 per cent of carbon in cast iron.

"So long as iron containing some 3 per cent of carbon remains in the fluid condition the composition is uniform throughout, and the carbon has no tendency to separate from the metal, except with very gray iron; in this case a layer of graphite, which often occurs in beautiful plates and is known as *kish*, may be formed. But when the molten cast iron is cooled to a temperature at which it begins to solidify, it may either retain the carbon and solidify in a relatively homogeneous form, called *white* iron; or it may, in solidifying, precipitate the greater part of the carbon in the form of small scales of graphite, which, being entangled by, and uniformly distributed through, the iron, impart to it a somewhat spongy nature, and produce the dark color and soft character met with in *gray* iron. When about half of the carbon is precipitated as graphite, and the rest retained in combination, the result is the production of dark gray portions in a matrix of white, and the iron is then said to be *mottled*.

"The condition which the carbon assumes on the solidification of the mass is dependent partly on the rate of cooling, and still more on the nature and quantity of the associated elements. In connection with the influence of cooling, cast iron obeys the laws which govern other solutions, for it is well known that slow cooling assists the production of crystals, and leads to the formation of crystals of larger size, while with rapid cooling both solvent and the substance dissolved may solidify together. In a similar manner slow cooling tends to produce graphitic carbon, and the slower the cooling the larger are the flakes of graphite which sepa-

rate. Some kinds of white iron may thus be rendered gray by slow cooling, while some kinds of gray iron may be made perfectly white by rapid cooling or 'chilling.' It is, however, only with intermediate irons that the rate of cooling produces a marked effect, for irons which are either very white or very gray cannot be changed in this manner. The influence exerted on the condition of the carbon by the other elements present in cast iron is of the greatest importance; thus manganese and chromium, which increase the solubility of carbon in iron, lead to a greater percentage of total carbon in the fluid metal, and when the iron solidifies this carbon is retained in solution, so that irons rich in manganese and chromium are white and no amount of slow cooling will alter this character. On the other hand, silicon and aluminum diminish the solubility of carbon in iron; if much of either of these elements be present in the fluid metal, it is capable of dissolving less carbon, and retains it with less energy when it solidifies; as a result the carbon is precipitated as graphite, and gray iron is produced. Just as irons which contain much manganese or chromium are permanently white, so metal rich in silicon or aluminum is permanently gray.

"The proportion of total carbon in iron to be employed for a given purpose is often of secondary importance; it is governed by furnace conditions, and by the proportion of other elements. A moderate alteration in total carbon, or in the graphite, will frequently have little effect on the physical properties of the product, while a small change in the combined carbon will profoundly alter the strength and hardness of the casting. *Probably no other constituent in cast iron is of importance equal to that of combined carbon, and the influence of the other elements is largely due to the effect they produce in increasing or diminishing the combined carbon.* The following percentages of combined carbon will usually be found suitable for the purposes specified:

	* Combined Carbon in parts of one per cent.
Extra soft siliceous gray iron	0.08
Soft cast iron	0.15
Maximum tensile strength	0.47
Maximum transverse strength	0.70
Maximum crushing strength	over 1.00

These figures are, however subject to some variation according to the size of the casting and the proportion of other elements. The hardness of the metal increases regularly with the increase of combined carbon."

64. Silicon in Cast Iron.—"All cast iron contains silicon, in quantities

* Chemical ingredients of iron and steel are always given in *hundredths of one per cent.* Thus "twenty carbon" and "eight phosphorus" signifies 0.20 and 0.08 of one per cent of each, respectively. Even the common workmen use these terms, though they may not always understand them. The word *point* is often added, as "twenty-point carbon."—J. B. J.

varying in ordinary cases from under 0.5 to over 4 per cent, while 'silicon pig' is made in the blast-furnace with from 10 to 18 per cent of silicon. No factor is of greater importance in determining the suitability of a sample of cast iron for any purpose in the foundry than its content of silicon, as this element is so constantly present, and its proportion is so variable, while the influence it exerts on the condition of the carbon present, and consequently on the hardness and fluidity of the metal, is so marked. It was formerly very generally held that silicon was injurious in all proportions, and the less there was present in iron for foundry purposes the better. It is true that Sefström had observed, long ago, 'that the carbon in gray iron in which much silicon exists, say from 2 per cent to 3 per cent, is wholly, or nearly so, in the graphitic state.' * A similar observation was made by Snelus in 1870, and was still more plainly stated by Ledebur in 1879. It was also known in the United States that certain irons from Ohio which were rich in silicon could be used as 'softeners' in foundry practice, and certain Scotch irons were in favor for similar purposes, though the reason of this was not understood. It may, however, be claimed that no general application of these facts, or accurate knowledge of the principles underlying them, existed before the researches of the author, on the 'Influence of Silicon on the Properties of Cast Iron,' published in 1885.† For the purpose of these experiments cast iron as free as possible from silicon was specially prepared by heating wrought iron with charcoal to a high temperature in closed crucibles. This was then remelted with a silicon pig containing about 10 per cent of silicon in proportions necessary to yield any desired composition. The trials were made with sufficient material to allow of proper mechanical tests being performed, and a graduated series of mixtures was prepared. The tension, compression, and ductility tests were performed by Professor A. B. W. Kennedy with the testing-machine at University College, London, while the hardness determinations were performed by the author with a weighted diamond point (see Chapter XVIII) as described in his paper on the 'Hardness of Metals.' ‡ The chemical analyses were checked by J. P. Walton, at that time chemist to the Glasgow Iron Company, Wishaw."

"The original pure cast iron was white, hard, and brittle; on adding silicon this became gray, soft, and strong; but with a large excess of silicon it once more became weak and hard. The results of the mechanical and chemical tests are shown graphically in Fig. 55, and it will be observed that the proportions of silicon corresponding to the various properties were as follows:

* Percy, p. 181.
† *Journ. Chem. Soc.*, 1885, pp. 577, 902.
‡ *Birm. Phil. Soc.*, Dec. 1886.

Maximum hardness.................................under 0.80 per cent.
" crushing strength............................about 0.80 "
" modulus of elasticity........................ " 1.00 "
" combined crushing and tensile strength; transverse strength.............................about 1.40 "
" tensile strength............................. " 1.80 "
" softness and working qualities,.............. " 2.50 "
" lowest combined carbon.......................under 5.00 "

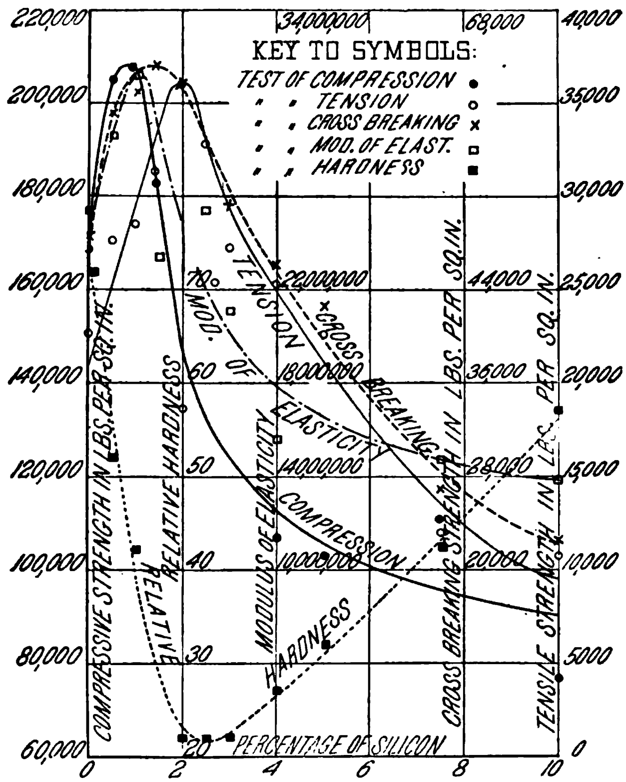

FIG. 55.—Showing the Influence of Silicon on the Strength and Hardness of Cast Iron. (From Turner's *Iron.*)

"It must be borne in mind that these values are only true for the author's experiments. Experience has since proved that these are approximately correct in other cases, and that the order is as above given; but in practice the size of the casting and the proportion of other elements will have an important influence."*

The influence of silicon on the shrinkage of cast iron, in various sizes up to 4 inches square, is well shown in Fig. 56. These results have been well

* See also Arts. 76 and 79.

established by Mr. W. J. Keep of Detroit. His results of transverse tests of strength and deflection, for varying proportions of silicon, are given in Figs. 57 and 58.

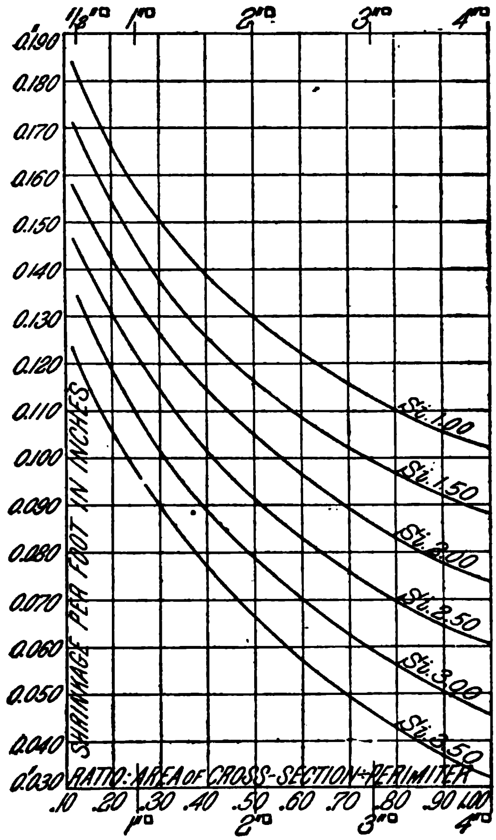

Fig. 56.—Showing the Influence of Silicon on the Shrinkage of Cast-iron Specimens of Various Areas of Cross-section. (Keep.)

"A small addition of silicon eliminates blowholes and produces sound castings. As soon as the metal is sound, with the least graphite, the greatest crushing strength is obtained; this condition also gives the maximum density. Further addition of silicon leads to the formation of graphite, diminishes the brittleness, and gives the greatest transverse and tensile strength. When the graphite increases beyond this point, the metal is divided by the interspersed graphitic material, and the strength and hardness decrease. The deflection also increases with the increase of graphite, but when the maximum separation of graphite has taken place any further addition of silicon causes stiffness or brittleness, and so diminishes the deflection. White iron shrinks during solidifying more than gray iron, while highly

siliceous iron shrinks still more than white. Hence on adding silicon to white iron the shrinkage is diminished, but an excess of silicon, on the other hand, leads to increased shrinkage. Shrinkage appears to closely follow the hardness of cast iron, hard irons almost invariably shrinking most; and as

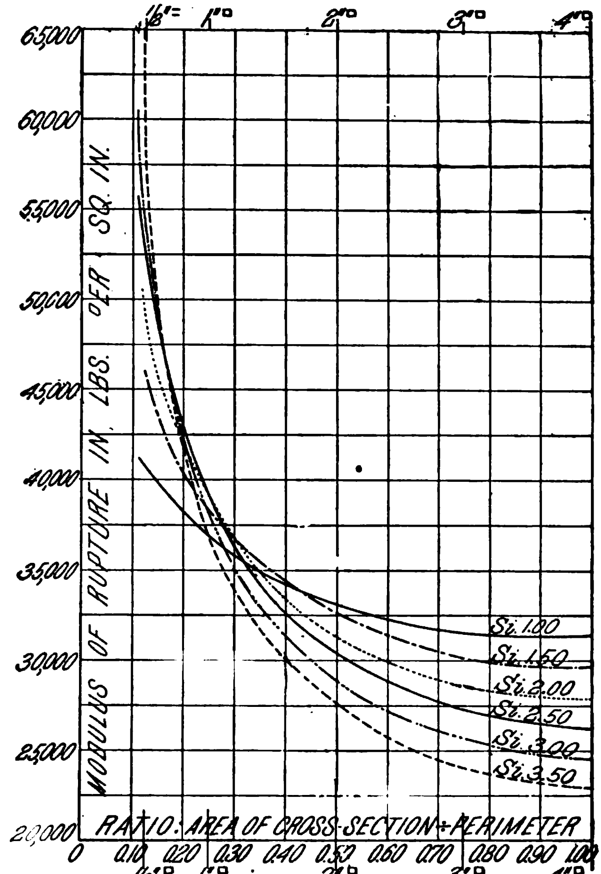

FIG. 57.—Showing Variation in Cross-breaking Modulus of Rupture of Cast Iron for Different Sizes of Bars and for Varying Percentages of Silicon. (Keep.)

both hardness and shrinkage depend upon the proportion of combined carbon, they may be regulated by a suitable addition of silicon."*

It has been shown by Mr. Keep that the influence of aluminum on cast iron is practically the same as that of silicon, equivalent effects being produced, however, with much smaller proportions of aluminum, as little as 0.1 per cent of aluminum causing the iron to become soft and graphitic. Since the same effect can be obtained by the use of silicon,

* *Trans. Amer. Inst. Min. Eng.*, 1888.

which is much cheaper, and since the action of the silicon is more uniform, because of the difficulty of controlling the effects of such small proportions of aluminum, the use of aluminum for this purpose is not likely to come into general use.

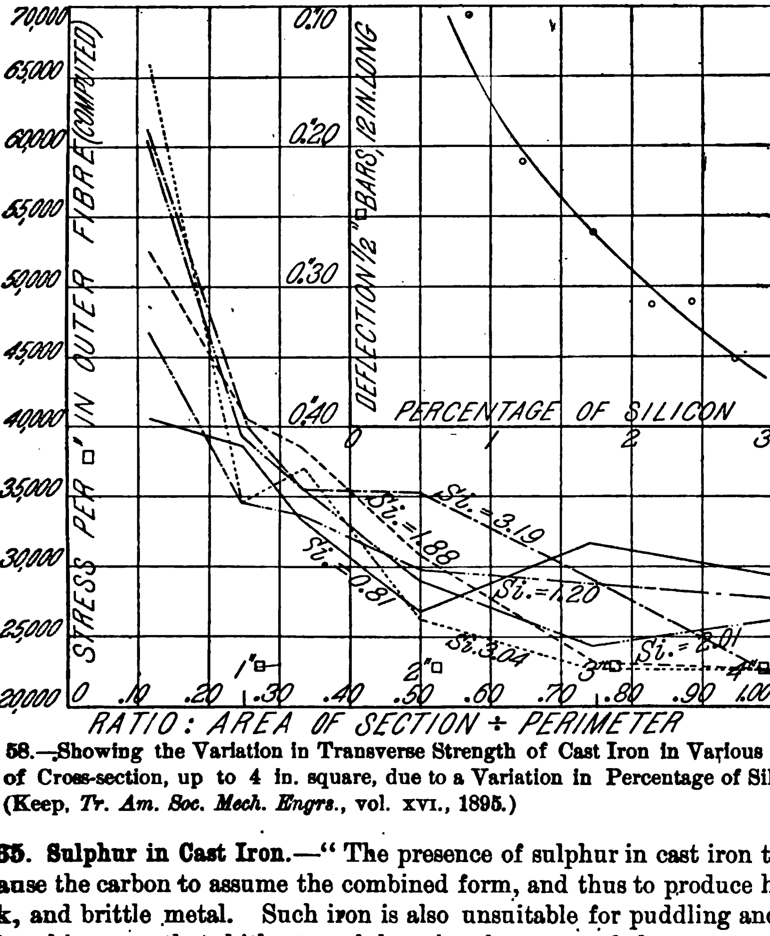

FIG. 58.—Showing the Variation in Transverse Strength of Cast Iron in Various Sizes of Cross-section, up to 4 in. square, due to a Variation in Percentage of Silicon. (Keep, *Tr. Am. Soc. Mech. Engrs.*, vol. XVI., 1895.)

65. Sulphur in Cast Iron.—"The presence of sulphur in cast iron tends to cause the carbon to assume the combined form, and thus to produce hard, weak, and brittle metal. Such iron is also unsuitable for puddling and for steel-making, so that hitherto sulphur has been regarded as a specially objectionable element. Foundry iron of good quality should not contain more than 0.15 per cent of sulphur."

66. Phosphorus in Cast Iron.—"The phosphorus which is present in cast iron exists in the form of phosphide, and is in great part eliminated with the excess of hydrogen as phosphoretted hydrogen, when the metal is treated with dilute sulphuric or hydrochloric acid. For many purposes, such as the manufacture of steel by either of the acid processes, or the production of wrought iron for conversion into tool steel, it is of prime importance that the proportion of phosphorus should be as low as possible, and the

maximum limit for such purposes is 0.06 per cent. It was formerly held that foundry iron should also be free from phosphorus, but the author has shown that cast irons of special strength always contain a moderate proportion of this element. If a large proportion of phosphorus be present, such as from 2 to 5 per cent, the metal is very fluid when melted, and takes an excellent impression of the mould. On this account such iron is sometimes employed for the production of very fine thin castings, but it cannot be used for any purpose where strength is required, as the presence of so much phosphorus induces great brittleness. The brittleness caused by phosphorus is so marked that a practical man can often approximately tell the percentage of phosphorus by the readiness with which the pig iron fractures when dropped on the pig-breaker. On the other hand, gray pig iron containing merely a trace of phosphorus, such as that from the best hematite or magnetite ores, is so soft and malleable as to be somewhat wanting in strength and soundness, and hence gives inferior results for rolls, columns, girders, and other purposes for which strength is necessary. In exceptional cases it is advantageous to have the phosphorus as low as 0.20 per cent in cast iron, but it is doubtful whether there is any advantage in going below this limit. For ordinary strong castings of good quality about 0.55 per cent of phosphorus gives excellent results. For the general run of foundry practice, where fluidity and softness is of more importance than strength, from 1 to 1.5 per cent of phosphorus may be allowed, but beyond this higher limit the further addition of phosphorus causes such brittleness as to lead to marked deterioration."

67. Manganese in Cast Iron.—"The proportion of manganese which is met with in iron produced in the blast-furnace ranges from a mere trace to upwards of 86 per cent, and, speaking generally, the higher the percentage of manganese the more valuable is the product, on account of the use of this element by the steel-maker. The physical properties of cast iron are not greatly altered so long as the manganese present does not much exceed 1 per cent, and larger proportions may be present in siliceous iron without producing the appearance in the fracture which is so characteristic of manganese. When about 1.5 per cent of manganese is present the iron is very appreciably harder to the tool, and is more suitable for smooth or polished surfaces. But when the amount of silicon is relatively small, and the manganese exceeds 1.5 per cent, a white iron is obtained with a glistening fracture showing flat crystalline plates, which, when very marked, leads to the application of the name 'spiegeleisen' or mirror iron, and which is too hard to be cut by cast-steel tools. Speigeleisen contains up to 20 per cent of manganese, but with higher proportions the grain becomes once again uniformly close and granular, and a material is obtained which exhibits a characteristic light-gray color, and which is so brittle that it may be readily pounded in an iron mortar. To these varieties the term 'ferro-manganese' is applied; while for some purposes an iron rich in both silicon and manganese, containing, for example, 10 per cent of silicon and 20 per cent of

manganese, is produced, and is known as 'silicon-spiegel' or 'silicon ferro-manganese.'

"From the examination of the tests conducted at Woolwich in 1858,* and numerous analyses of selected samples of cast iron of special strength, the author concluded that the presence of some manganese was rather beneficial than otherwise in foundry practice, though probably any benefit ceases when the proportion of manganese is much greater than 1 per cent.† The good effect of manganese appears to be twofold; by its own action it leads directly to a measure of hardness and closeness of grain which is beneficial, while indirectly it is useful in preventing the absorption of sulphur during remelting.

"The effect of manganese when alone is always to harden cast iron, and yet cases have come under the author's notice in which in actual practice ferro-manganese has been added in small quantity to molten metal in a foundry ladle, with the result that the iron has been very much softened and improved. The reason for this doubtless lies in the fact that manganese counteracts the effect of sulphur and silicon, tending to eliminate the former and neutralize the latter, and so, where common iron is employed, it sometimes happens that ferro-manganese may be used as a softener. The hardness, however, generally returns if the iron be remelted, as the manganese is oxidized and more sulphur absorbed.

"Manganese has in this way been employed as a softener. A remarkable effect is produced on the properties of hard cast iron by adding to the molten metal, a moment before pouring it into the mould, a small quantity of powdered ferro-manganese, say 1 lb. of the latter to 600 lbs. of cast iron. As a result of several hundred carefully conducted experiments the transverse strength was increased 30 per cent, the shrinkage and depth of chill decreased about 25 per cent, while the combined carbon was diminished by about one half.‡ These observations accord with those made by the author, though in all probability their success depends, as above explained, on the peculiar composition of the cast iron used."

68. Grading of Pig Iron.—"For commercial purposes pig iron is classified or 'graded' according to the appearance of the fractured surface, the first member of the series being taken as the most open-grained gray iron, while white iron is taken at the other extremity."

"In the southern parts of the United States the following method of grading pig iron into nine numbers was adopted in 1889:§

1. No. 1 Foundry.	4. No. 1 Soft.	7. Gray Forge.
2. No. 2 Foundry.	5. No. 2 Soft.	8. Mottled.
3. No. 3 Foundry.	6. Silver-gray.	9. White.

* *Report, Cast-Iron Experiments*, 1858.

† *Inst. Journ.*, 1886, vol. I. p. 185.

‡ *Jour. Franklin Inst.*, Feb. 1888.

§ *Iron Age*, vol. XLII. p. 498.

"The following analyses, published by G. L. Luetscher, of the pig iron made from the ore of Red Mountain, Alabama, will serve to illustrate the composition of the different grades of Southern iron: *

	Silver Gray.	No 2 Soft.	No. 1 Soft.	No. 1 Foundry.	No. 2 Foundry.	No. 3 Foundry.	Gray Forge.	Mottled.	White.
Graphitic carbon......	3.13	3.48	3.53	3.49	3.55	3.48	3.00	2.11	.10
Combined carbon.....	.02	.03	.03	.07	.07	.10	.57	1.22	2 92
Silicon..............	5.5	3.5	3.75	3.15	2.40	2.20	1.50	1.35	.95
Sulphur..............	trace	.004	.005	.005	.024	.025	.06	.125	.30
Phosphorus..........	.68	.68	.68	.68	.68	.64	.64	.64	.64
Manganese...........	.25	.26	.27	.25	.22	.21	.19	.14	.10

"It will be observed that the silicon regularly decreases, with one slight exception, from 5.5 per cent with silver gray, to 0.95 per cent with white iron. At the same time the sulphur and combined carbon increase together from mere traces in silver-gray iron to 0.3 per cent of sulphur, and nearly 3 per cent of combined carbon in white iron. The phosphorus is slightly lower with the closer grades. These differences are exactly such as are noticed with similar grades in the United Kingdom, the most noticeable difference being the remarkably small quantities of sulphur met with in the open-grade American iron. That American foundry irons have an unusually low percentage of sulphur is a fact which is supported by the results of numerous analysts, and which has not yet been satisfactorily explained."

FOUNDRY PRACTICE.

69. "**The Cupola** is in by far the most general use for remelting iron. A cupola is a small blast-furnace, of which there are many varieties employed; they are generally circular in section, and are driven with low-pressure blast at or near the atmospheric temperature. The fuel used is generally hard coke, though occasionally gaseous fuel or charcoal is employed. Usually the melted metal collects at the bottom of the cupola, and is tapped off at intervals; in some cases separate receivers are adopted.

"When coke is used the fuel consumption varies from about 1¼ to 2½ cwts. per ton of iron melted, being greater with small outputs on account of the loss due to heating the cupola with each charge. A small quantity of limestone is usually added, as it fluxes off the silica added in the form of sand adhering to the pigs, or produced by the partial oxidation of the silicon in the iron; it combines with the ash of the coke, and also diminishes the amount of sulphur which is absorbed from the coke by the iron.

"The blast, which is not heated, is driven by means of a fan, or more usually by a blower, the pressure being only a few ounces per square inch. In the ordinary form of cupola the blast is introduced through one or more tuyeres in a single row around the zone of fusion. In Ireland's cupola,

* *Inst. Journ.*, 1891, vol. II. p. 245.

which was introduced about 1860, two rows of tuyeres are employed, and the cupola is provided with boshes like a blast-furnace. The object of the upper row of tuyeres is to insure more complete combustion of the carbonic oxide, which otherwise passes through the charge unburned."

70. Influence of Remelting.—" It is observed that when cast iron is remelted it becomes harder and more close in texture; if the metal operated be soft, the casting is stronger than the original iron; but when hard iron is used, it becomes still harder, and weak, like ordinary foundry scrap. There has long been an impression that remelting improves cast iron, but that this is not so is proved by melting the metal in a carefully covered crucible, where no change in composition takes place, and the properties of the iron are unaltered. In some experiments by Sir W. Fairbairn,* a sample of No. 3 Eglinton gray iron was remelted in an air-furnace 18 times, test-bars being cast at each melting, and it was found that the iron improved up to the twelfth melting, and afterwards rapidly deteriorated. Other experiments were performed shortly afterwards, in connection with the manufacture of cast-iron ordnance, in which marked improvement was noticed on remelting cast iron, and keeping it for a longer or shorter period in a state of fusion. No explanation of these effects was given, but the experiments were referred to in numerous text-books, and led to the belief that remelting *per se* was beneficial, though it was observed that the number of remeltings required to produce the best effect varied largely with different samples.

" By the kindness of Professor Unwin, who assisted in Sir W. Fairbairn's experiments, the author was supplied, more than thirty years after the tests were made, with samples of the test-bars, and was enabled by their analysis to clear up some of the difficulties which had surrounded the subject.† The results of the author's analyses were as follows:

No. of Melting.	Total Carbon.	Combined.	Silicon.	Sulphur.	Manganese.	Phosphorus.
1	2.67	0.25	4.22	0.03	1.75	0.47
8	2.97	0.08	3.21	0.05	0.58	0.53
12	2.94	0.85	2.52	0.11	0.33	0.55
14	2.98	1.31	2.18	0.13	0.23	0.56
15	2.87	1.75	1.95	0.16	0.17	0.58
16	2.88	....	1.88	0.20	0.12	0.61
18	....	2.20	....	....	....	....

" It will be noticed that, owing to the oxidizing effect of remelting, the proportion of silicon steadily diminished, while sulphur was at the same time absorbed from the furnace gases. The natural effect due to these changes was produced upon the condition of the carbon, which, instead of being almost wholly graphitic, became nearly all combined, thus producing a hard, white iron, which was deficient in tenacity, and brittle. The

* *B. A. Report*, 1853, p. 87.

† *Journ. Chem. Soc.*, vol. XLIV, 1886, p. 493.

elimination of manganese with the silicon, the increase in the percentage of phosphorus due to its concentration in a smaller quantity of metal, and the initial increase of total carbon for a similar reason, are all in accordance with what is observed whenever iron is melted in the air, and when the resulting slag is not strongly basic.

"The physical effects produced when cast iron is remelted are thus merely indications of chemical changes which have taken place in the material, while the nature of these changes, and hence the effect produced by remelting, will vary with the composition of the iron employed and the oxidation to which it is subjected.

"In Sir W. Fairbairn's experiments the metal was melted in an air-furnace, but in ordinary practice a cupola is employed. Here the oxidation is greater, while as the iron melts in contact with the fuel it more readily absorbs sulphur. As a consequence, though the changes which take place are of the same kind, and follow the same order as that previously given, the effect of each melting is more marked. This is illustrated by the following analyses,* from experiments conducted by Jüngst in the Imperial Foundry at Gleiwitz:

	1st Melting.	2d Melting.	3d Melting.
Carbon, graphitic	2.73	2.54	2.08
Carbon, combined	.66	.80	1.28
Silicon	2.42	1.88	1.16
Sulphur	.04	.10	.20
Phosphorus	.31	.30	.28
Manganese	1.09	.44	.36

71. Moulds.—"The size, shape, and character of the moulds employed in an iron foundry depend upon the class of work in hand; they may be conveniently divided into the following four classes:

1. Green-sand;
2. Dry-sand;
3. Loam;
4. Chills.

1. "*Green-sand moulds* are made of moulding-sand, which is first uniformly damped, so as to make it adherent, and is lightly rammed around a pattern to obtain the required shape. For common castings, especially when of large size, open sand is often used; but for the majority of purposes the sand is contained in boxes, which in the United Kingdom are usually of cast iron, though wooden moulding-boxes are generally used in the United States. Usually there are two boxes, upper and lower, the pattern or patterns being placed partly in each box, and the 'gate' or opening for the entry of the metal being commonly in connection with the middle of the

* *Inst. Journ.*, 1885, vol. II. p. 645.

castings. Where a hole or passage is required in the casting, a 'core' is employed; this generally consists of sand, moulded into the necessary shape, and supported by iron wire or other suitable means. The patterns are generally of wood, and if of intricate forms, are made in parts designed to allow of their removal from the mould; the several parts are kept in position by suitable pins. Green-sand moulding is the process most generally adopted, as it is rapid and cheap; it involves the use of no expensive plant, and is specially suitable for the production of a large number of articles of similar form. Machine moulding is employed by manufacturers who have a considerable demand for one class of work, and in such cases sand-moulding machines are coming steadily into favor, though they can never replace hand work in a general foundry.

2. "*Dry-sand moulds* are made of a loamy sand which, after being roughly moulded into shape, is dried by heat, and then carefully finished with the tool. The mould is sufficiently soft to be readily cut, though rigid enough to retain its shape when the molten metal is poured into it. Such moulds have the advantage of giving sounder castings, as they evolve less gas, while where a single casting is needed they save money, as no pattern is required. When, however, a pattern has once been prepared, green-sand moulds are much cheaper.

3. "*Loam-moulds* are more particularly employed for curved or spiral surfaces of large size, such as sugar-pans, 'copper' boilers, soda-pans, water-pipes, etc. The outer part of the mould is either built up of brickwork, held in place with iron ties; or where a number of similar articles is required, an iron casing is employed. The inner surface of the mould is made of loam, which is laid on by the trowel, and worked by the hand, and usually faced with some carbonaceous blacking. The whole is then carefully dried before use, one of the most general methods being by the introduction of a flame of gas into the interior. Such moulds can, of course, only be employed for one casting, and the labor and cost of loam-moulding is much greater than that of green-sand.

4. "*Chills* are used when it is desired to produce a casting the outside of which is unusually hard. The iron used is generally a close-grained gray, and this is converted into white iron where it comes in contact with the cold side of the mould during solidification. A familiar example of the use of chills is met with in the production of chilled rolls and car-wheels. Rolls are cast on end, with a good head of metal, so as to give soundness, while as the shanks of the roll are required to be turned to size, these are cast in sand, and are, therefore, relatively soft. The intermediate part of the mould, in which the barrel of the roll is cast, is made up of a number of large annular rings of cast iron resting one upon another. These are not used cold, or a violent explosion would take place when the hot metal came in contact with the cold, and therefore probably slightly damp, chilling surface. The mould is, on this account, heated to a temperature of about 150° to 200° C. before the metal is introduced, and the iron is caused to enter

from the bottom, and in an oblique direction. By this means a circular motion is imparted to the metal, and thus, as it rises, it collects all dirt and impurities on its surface, and so fills every crevice of the mould."

72. Moulding-sand.—"The proper selection and preparation of moulding-sand has an important influence on the appearance and quality of the castings produced in the foundry. The mould must be capable of retaining the fluid metal in every direction, but at the same time it must allow of the free passage of the air which is collected, and the gases which are generated when the mould is filled with hot iron. It must give to the casting a smooth, clean surface, and hence must neither act upon, nor be affected by, the fluid metal at the high temperature at which they are brought in contact; the higher the temperature is that is necessary to retain the metal in the perfectly fluid condition, the greater is the difficulty of complying with this condition. Thus moulds for cast iron require more careful preparation than those for brass, while those to be employed for steel castings require still more careful attention. Moulding-sands consist chiefly of silica, together with variable proportions of alumina, magnesia, lime, and other metallic oxides; coal-dust is also frequently added in small quantity. The higher the proportion of silica the more refractory the sand becomes; but it is then apt to be wanting in cohesion, and to be difficult to mould, while the moulds crack in drying, or are injured by the flow of metal. Alumina and magnesia impart cohesion and plasticity, though excess, especially of alumina, causes it to be less refractory. Magnesia is refractory, forming a good cement for siliceous sand, but when present in quantity it renders the mould less porous. Lime and other metallic oxides render sand less refractory, and should be avoided as far as possible. If the lime be present as carbonate, gas will be given off at high temperatures, and will produce rough surfaces in the casting; while if it be present as silicate, it will cause the sand to adhere to the surface of the hot metal. According to Kohn,* a suitable composition for green-sand moulding is approximately as follows: Silica, 92 per cent; alumina, 6 per cent; oxide of iron, 1.5 per cent; and lime 0.5 per cent; while sand for stove-dried moulds is usually richer in alumina and oxide of iron. According to the same author, a composition largely used in steel-works for moulding purposes is prepared from Sheffield ganister, which is mixed with sufficient magnesia and alumina to give a product containing about 85 parts of silica, 5 to 10 of alumina, and 5 to 10 of magnesia.

"In addition to the sand being of the right chemical composition, which condition affects its plasticity and refractory nature as above indicated, it is also necessary that it should be of proper degree of fineness, as when the particles are too coarse the surface of the castings is inferior, and the sand is wanting in cohesion, while when the sand is unusually fine it is unsuitable for large castings, as the gases cannot so readily escape."

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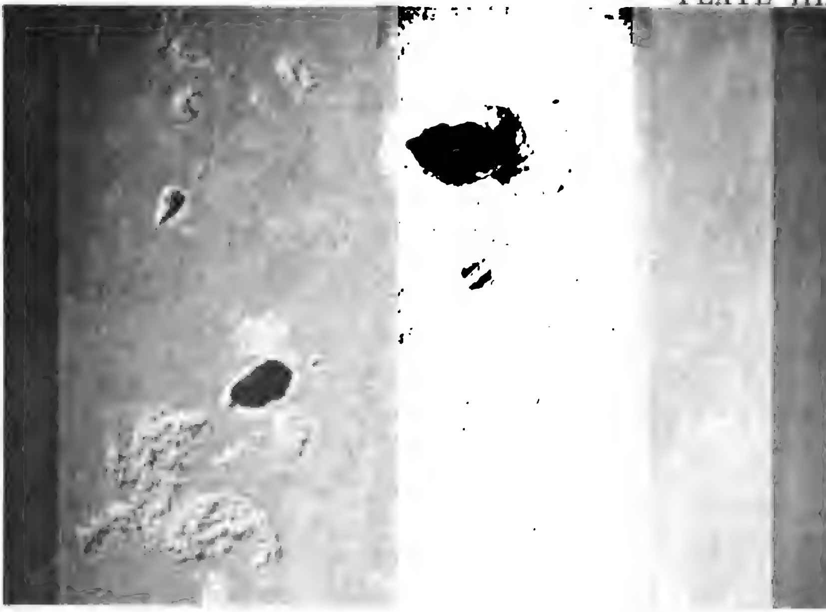
* *Iron Manufacture*, p. 55.



HIDDEN DEFECTS IN A CAST-IRON COLUMN DISCOVERED AFTER THE BUILDING HAD BEEN ERECTED.

FAILURE OF A STEEL STAND-PIPE DUE TO BRITTLE BESSEMER-STEEL PLATES IN THE TWO LOWER COURSES, THE INITIAL RUPTURE SHOWN IN THE FOREGROUND.

The tensile stress on the net section at time of failure was only one fifth the ultimate strength of the metal. (Examined and photographed by the author.)

73. Effect of Size and Shape.—"The strength and solidity of a casting are affected by the bulk of metal employed, and by the form of casting made. Thus if a sample of pig iron which would be suitable for a casting of small size be employed for making very heavy work, it will be found that owing to the slower cooling in the latter case the grain of the metal becomes much more open, and the strength is proportionally diminished; on the other hand, if the same metal were used for very small castings, the chilling in the mould would tend to make the product close and hard, and in many cases this would be so marked as to make the castings quite brittle. The grade of the iron used must therefore depend upon the size of the casting to be made, the general rule being that a closer-grained or less siliceous iron must be used for large than for small castings. At the same time it is generally found that the strength of a large casting per unit of area is somewhat less than that of a smaller one, since the closeness of grain is usually, though not always, associated with increased tenacity.

"It is also very important that in large castings, where strength is required, no sharp or re-entering angles should occur, as these in all cases lead to the formation of planes of weakness in the casting. When the metal cools in the mould a crystalline structure is developed, the crystals forming at right angles to the cooling surface. If this cooling surface be curved, the crystals interlace so as to yield a strong casting of uniform structure, while, on the other hand, whenever a sharp angle of curvature takes place a plane of weakness is the result."

Cast-iron columns, used in architectural construction, are commonly cast in a horizontal position, thus offering a very poor means of escape for loosened sand, cinder, etc., which foreign matter often collects on the upper side and greatly weakens the column. Furthermore, these are often covered by a thin coating of iron. Such faults, found in many columns which had been accepted and used * in a public building in St. Louis, are shown in Plate III.

74. Shrinkage of Cast Iron.—"Although cast iron, especially when very gray, expands at the moment of solidification, and thus gives a sharp impression of the mould, the subsequent cooling from a red heat to the ordinary temperature leads to a still greater contraction, and the net result is that the casting is always smaller than the pattern from which it is made. For this reason it is usual in pattern-making to allow about $\frac{1}{8}$ of an inch per foot for shrinkage, and if the casting is required 1 foot long, the pattern is made 1 foot and $\frac{1}{8}$ of an inch in length. The shrinkage in castings is, however, by no means a constant quantity, but varies with the relative dimensions of the castings and with the character of the metal used; as much as $\frac{1}{10}$ of an inch per foot being allowed when casting beams, and only $\frac{1}{32}$ with large cylinders. Not unfrequently much loss and inconvenience is occasioned in foundry work by variations in the shrinkage, caused by altering the shape or proportions of a pattern, or by the use of a different variety of iron.

* They were removed after the faults were discovered.

"In the author's experiments on cast iron it was noticed that silicon pig shrank most in the mould, though no accurate determinations of shrinkage were made. The subject has since been carefully investigated by W. J. Keep, of Detroit, whose experiments embody the whole of the trustworthy data available, and who measures shrinkage by casting bars in sand between iron chills 12½ inches apart. The contraction is carefully measured by means of graduated wedges which are inserted between the ends of the cold bar and the iron chill in which the bar was cast. Mr. Keep concludes that when silicon varies, and other elements do not vary materially, castings with low shrinkage are soft, and that as shrinkage increases, hardness increases in almost, if not exactly, the same proportion. For ordinary foundry practice the scale of shrinkage agrees with the scale of hardness, so long as sulphur and phosphorus do not vary over wide limits. This is an important fact; and as shrinkage tests are very easily performed by an ordinary workman, the subject is worthy of more attention than it has hitherto received." *

"It is stated that charcoal iron has usually a melting-point which is considerably higher than that of less pure iron made with coke. Charcoal iron, therefore, sets more quickly in the mould, and contracts more, so that an extra allowance for shrinkage must be made in the patterns employed." †

THE MECHANICAL PROPERTIES OF CAST IRON.

75. "Hardness of Cast Iron.—The hardness or softness of cast iron is in many instances of the greatest importance, as the metal has to be turned, planed, filed, or otherwise worked with tools; hence a number of methods have been devised at various times, with the object of determining relative hardness. In the older form of apparatus, such as was used by the American Ordnance Commissioners in 1856, an indentation was made in the surface of the metal to be tested. By determining either the force required to make a hole of a given size, or, on the other hand, the size of the indentation produced by a given force, a measure of hardness was sought to be obtained. Such a method is, however, erroneous unless the tenacity of all the specimens to be examined is the same, as otherwise a deeper hole will be produced in the weaker metal, irrespectively of hardness.‡ In the author's researches a weighted diamond was employed for determining the hardness of cast iron, and the results obtained with increasing proportions of silicon are graphically represented in Fig. 55. When very little silicon was present the metal was extremely hard owing to the large proportion of combined carbon, while when sufficient silicon had been added to convert the greater part of the carbon into the graphitic form the maximum softness was obtained. With further additions of silicon the metal became harder owing to the hardening effect of silicon itself, and for this reason an excess of silicon, beyond about 3 per cent, is injurious to the working qualities of the metal."

* W. J. Keep, *Silicon in Cast Iron*, p. 22. † Kohn, *Iron Manufacture*, p. 57.

‡ The French Commission recommend this test for hardness, using impacts of known value in place of static pressure. See Chap. XVIII.—J. B. J.

76. Hardness and Strength of Cast Iron.—"When cast iron has to be turned or otherwise worked the hardness is of considerable importance, while in some cases smoothness of surface and general perfection of the casting are of the utmost moment. Hard cast iron is brittle, deficient alike in crushing, transverse, and tensile strength, and seldom gives smooth clean castings. With metal which is a little less hard the maximum crushing strength is obtained; while on rendering it a little softer, or, as the workman would call it, 'moderately hard,' the maximum transverse strength is observed. With slightly softer cast iron the highest tensile tests are obtained, while still softer metal works with the utmost facility, though it is deficient in strength. It will be seen, therefore, that when the general connection between hardness and strength has been fully grasped, the iron-founder requires only the information how to harden or soften his metal at will, by the use of silicon or other agents, to be able to produce castings in which crushing, transverse, or tensile strength shall predominate as desired, or in which softness and fine surfaces shall be the most characteristic feature.

"There is a somewhat prevalent idea among founders that if considerable strength is required a hard iron must be employed. Doubtless this is to some extent true in connection with crushing and transverse tests, but is certainly not correct with tensile strength. In all specimens of exceptionally high tensile strength examined by the author the metal was a soft good working iron, specially suited for engineers' purposes. In the accompanying table is a summary of the author's results on the tenacity and hardness of cast iron, as affected by alterations in the proportion of silicon.* The working qualities of the specimens are also given, and it will be seen that the hardness as determined by the sclerometer agrees very closely with the observations of the workman. It will be noticed, however, that hardness and tensile strength do not vary together, but on the contrary high tensile strength is met with in the softer irons."

TABLE I.—INFLUENCE OF SILICON ON THE HARDNESS AND TENACITY OF CAST IRON.

No.	Silicon per cent.	Tensile Strength in Lbs. per Sq. In.	Hardness by Sclerometer.	Working Qualities, as determined by the Workman.
1	0.19	22,700	72	Very hard indeed.
2	0.45	27,600	52	Very hard, though not so hard as No. 1.
3	0.96	28,500	42	Hard, though softer than No. 2.
4	1.96	35,200	22	Good, sound, ordinary, soft-cutting iron, of excellent quality.
5	2.51	32,800	22	Rather harder than No. 4.
6	2.96	27,400	22	Like No. 4.
7	3.92	25,800	27	Like No. 6, but rather harder.
8	4.75	22,700	32	Rather harder than No. 7, though not unusually hard.
9	7.37	12,000	42	Still harder, cutting very like No. 10.
10	9.80	10,600	57	Hard-cutting iron, though still softer than No. 1.

* *Journ. Chem. Soc.*, 1885.

77. Crushing Strength.—"Cast iron possesses an exceptionally high crushing strength, and for the majority of purposes the founder relies upon this, and does not perform special tests. Usually the tensile strength is not above one sixth of the crushing strength; hence if power to resist a tensile force is assured, the crushing strength is usually sufficient for ordinary work. In performing compressive tests it is necessary to have perfectly parallel surfaces, and to bed the specimen as true as possible, otherwise the results will be low."

As shown in Articles 19 and 22, the height of a crushing-test specimen of such a material as cast iron should be not less than about twice its least lateral dimension. Test specimens of cast iron are usually cylinders which have been turned up in the lathe, and hence the length of such a specimen should be not less than twice the diameter. The following table of values contains the results of a great many tests of cast iron in compression on short cylinders, the dimensions being usually one inch in diameter and from two and one-half to three inches high.

TABLE II.—CRUSHING STRENGTH OF CAST IRON.

Experimenters.	Pounds per Square Inch.			Authorities.
	Max.	Min.	Mean.	
Hodgkinson..........	146,000	82,000	107,000	Fairbairn, *Iron*, 1869, p. 218.
Hodgkinson (1849)....	121,000	55,000	86,000	Pole, *Iron Construction*, p. 84.
Woolwich (1858).....	140,000	44,500	91,000	*Report*, 1858, p. 2.
Fairbairn............	215,000	92,000	......	*B. A. Report*, 1853, p. 87.
Turner..............	207,000	77,000	......	*J. Chem. Soc.*, 1885, p. 907.

"The average crushing strength of British cast iron is thus about 90,000 lbs. per square inch; exceptionally, results so low as 45,000 lbs. have been observed, while, on the other hand, a strength of upwards of 200,000 lbs. has been produced in some instances. In the above experiments no special pains were taken to produce an iron possessing a high crushing strength; on the contrary, only such irons were taken as were met with in commerce. In the light of modern researches, iron could doubtless be produced with a crushing strength of 225,000 lbs. to the square inch, while a strength of 150,000 lbs. could, if necessary, be regularly assured. A series of sketches illustrating the fractures of test-pieces with different proportions of silicon, when subject to a compressive force, are given in the author's paper on 'Silicon in Cast Iron.' * The samples were prepared by the author, and the mechanical tests performed by Professor Kennedy at University College. From these experiments it is probable that the maximum crushing strength would be obtained with about 0.75 per cent of silicon and 2 per cent of combined carbon."

* *Journ. Chem. Soc.*, 1885, p. 909.

78. Transverse Strength.—" As before stated, the maximum transverse strength is obtained with metal a little softer than that which possesses the highest crushing strength. Transverse strength depends, at least in part, on the power to resist both a crushing and a tensile force; hence transverse strength is intermediate between crushing and tensile so far as the character of the iron is concerned. This combination of properties imparts to the metal characters which are most valuable in certain cases. For transverse tests many shapes and sizes of test-bar have been adopted, and, for scientific purposes, the results so obtained are converted by calculation into breaking-stress on the extreme fibres in pounds per square inch, which is called the modulus of rupture. See Art. 33. For a load in the centre of a rectangular bar we have $f = \frac{3}{2}\frac{Pl}{bh^2}$, where

f = modulus of rupture in cross-breaking;
P = breaking load at centre;
l = length of bar in inches;
b = breadth of bar in inches;
h = height of bar in inches.

TABLE III.—MODULUS OF RUPTURE OF CAST IRON.

Experimenters.		Modulus of Rupture in Pounds per Square Inch. $f = \frac{3}{2}\frac{Pl}{bh^2}$.	Authorities.
Robert Stephenson, 1847...	Max..	58,000	Pole, *Iron for Construction*, p. 88.
	Min..	37,000	
Hodgkinson and Fairbairn.	Max..	47,500	Box, *Strength of Materials*, p. 186.
	Min	29,500	
	Mean.	37,000	
Woolwich, 1858...........	Max..	42,500	*Report*, p. 2.
	Min..	9,700	
	Mean.	26,700	
Fairbairn, 1853..............	Max.	56,000	*B. A. Report*, 1853, p. 87.
Turner, 1885.................	Max.	63,500	*Inst. Journ.*, 1886, p. 1.

" It will be noticed that the modulus of rupture varies from the exceptionally low value of 9700 lbs. to 63,500 lbs. The average for common iron is about 30,000, while 45,000 is required for better-class castings. For specially good work some South Staffordshire founders can produce a strength of 60,000 with tolerable regularity. In performing transverse tests, care should be taken to avoid even the slightest twist on the specimen, and the weights used should be added very gradually, otherwise low and irregular results are obtained. The size of bar used has also an influence on the strength (as shown in Figs. 57 and 58), the smaller sectional areas giving much higher values. It should be remembered that the strength of a test-bar does not accurately represent the strength to be expected in the casting, if the size of the latter, and the circumstances of pouring, do not pretty closely agree with those of the test-bar itself."

79. Tensile Strength.—" In many of the less important foundries tensile tests are omitted, but in the better works such tests are generally performed, and appear to be growing in favor. It was shown by the American Ordnance Experiments (1856) that the tenacity of cast iron usually serves as a guide to its mechanical value, and practical experience quite confirms this view. Tensile test-pieces are of various forms: they are sometimes used with the skin on, at others the surface is carefully turned; sometimes small pieces are cast separately, while other founders cast the pieces on to the object which is being made. At Rosebank Foundry, Edinburgh, the practice is to cast a test-piece on to the top and bottom of each important article; these pieces are afterwards broken off, and carefully turned down to a suitable size before breaking. Such a method is calculated to give a result very nearly approaching what may be expected in the casting itself; for not only is the test-piece of the same composition as the casting, but it is also cast under as nearly as possible the same conditions as to temperature, pressure of metal, and rate of cooling, all of which have a considerable effect on the strength of the product.

" The following table condensed from a paper by the author will serve to illustrate the results obtained by different observers: *

TABLE IV.—TENSILE STRENGTH OF CAST IRON.

Experimenters.	Pounds per Square Inch.			Authorities
	Max.	Min.	Mean.	
Minard and Desnormes, 1815....	20,300	12,200	16,000	*Tredgold*, 4th Ed., p. 230.
Hodgkinson and Fairbairn, 1837.	22,000	13,400	16,800	*B. A. Report*, 1837, p. 339.
" " " , 1849.	23,500	....	15,300	Pole, *Iron Construction*, p. 79.
Woolwich, 1858..............	34,800	9,200	23,300	*Report*, 1858, p. 2.
Turner, 1885..................	35,200	10,600	....	*J. Chem. Soc.*, 1885, p. 580.
Rosebank, 1886...............	40,800	....	....	

" It will be seen that the highest tensile strength of British iron above recorded (40,800 lbs.) was obtained in the experiments at Rosebank Foundry in 1886. The average tensile strength obtained by earlier experimenters was about 16,000 lbs., while in 1858 the mean was raised to 23,000 lbs. This increase represents a real improvement in the metal tested, and was due to a selection of the more suitable irons as a result of increased knowledge. Foundry practice has since improved, and some engineers now stipulate that a bar one inch in section shall be capable of bearing a weight of 22,400 lbs. for twenty-four hours without fracture, and this apparently severe test has been complied with. Contracts are now satisfactorily executed in which a minimum strength of 27,000 lbs. per square inch is required, and to produce this nothing but Cleveland iron is employed. The author has also succeeded in regularly producing an iron of excellent working qualities, with a tensile

* *J. S. C. I.*, vol. v. p. 289.

strength of 29,000 lbs. per square inch, from a mixture costing under two pounds (ten dollars) per ton, and consisting of cast-iron scrap and siliceous iron. This is a striking instance of the value of combined chemical and mechanical knowledge to the iron-founder.

"In foreign cast iron some tensile strengths have been recorded which have not yet been equalled in Britain, though probably these results are to be regarded as quite exceptional. Thus Professor Ledebur records a tensile strength of 42,700 lbs. per square inch with German iron,* while the American Commission on Metal for Cannon, in 1856, obtained a maximum of 46,000 lbs., and at the Wassiac furnaces, New York, 47,500 lbs. have been obtained.† Much difference of opinion has been expressed as to the value of tensile tests for cast iron, as the metal is now never used in tension. Professor Ledebur, who is probably the best authority on this subject in Germany, states that tensile tests should always be made; and the author's experience leads to the conclusion that where a complete system of tests, such as that of W. J. Keep, cannot be adopted, no other test affords so good an indication of the value of the metal, as cast iron with high tensile strength is almost invariably soft, sound, and fluid. In the following table seven analyses by the author of samples of cast iron of unusually high tensile strength are given, together with the results obtained at Woolwich in 1856, and at Wassiac. Full details of the preparation of these samples are given in the original paper."‡

TABLE V.—COMPOSITION OF CAST IRON HAVING A HIGH TENSILE STRENGTH.

	Woolwich Experiments, 1856, Average.	Silicon Experiments, 1885.	Rosebank Irons, 1886.				Dumbarton Irons.		Wassiac Iron.	Average.
Tensile Strength Pounds per sq. in.	....	35,000	40,700	38,200	37,200	36,700	37,000	34,000	41,200	37,500
	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent	Per Cent
Graphitic Carbon	2.59	1.62	....	....	...	....	2.90	2.60	2.31	....
Combined Carbon	....	0.56	0.36	0.58	0.52	0.40	0.32	0.30	0.78	0.475
Silicon	1.42	1.96	1.29	1.50	1.18	1.33	1.84	1.63	1.31	1.434
Phosphorus.....	0.39	0.28	0.56	0.47	0.41	0.70	1.09	1.10	0.29	0.587
Sulphur.........	0.06	0.03	0.06	0.07	0.06	0.05	0.14	0.12	0 08	0.074
Manganese	0.58	0.60	1.00	1.00	1.33	0.65	1.38	1.29	1.51	1.037

"The average composition shown in the above table may be regarded as typical for good cast iron when the maximum strength is desired, together with soundness and good working qualities. By increasing the silicon the

* *Inst. Journ.*, 1891, vol. II. p. 252.
† *Inst. C. E.*, vol. LXXIV. p. 378.
‡ *J. S. C. I.*, vol. VII. p. 200

metal becomes more soft and fluid, while by diminishing the silicon the transverse and crushing strength, together with the tendency to chill, are increased."

MALLEABLE CAST IRON.

80. The Product Defined.—White cast iron, or that which has its carbon all in the combined form, can be made "malleable," or somewhat ductile, and nearly doubled in strength, by a process of annealing, by which the carbon separates from the iron without forming a mesh or matrix in which the remaining iron crystals are imbedded and surrounded, as is the case in ordinary gray cast iron. It has hitherto commonly been assumed that this change in the iron was effected by means of a decarbonizing agent (oxide of manganese, or iron oxide scale), which caused the carbon to leave the iron and enter into combination with the surrounding materials, thus making the iron malleable. But Mr. H. R. Stanford has shown,* confirming the results of Ledebur—

1. That only about 10 or 20 per cent of the total carbon is lost in the process; and,

2. That the same results are effected when the castings are packed in clean river sand—at least so far as the interior portion of the cross-section is concerned. It further appears from his investigations that the carbon of this interior portion is simply changed from the combined to the graphitic form (Ledebur's "temper-carbon") at a bright cherry-red, the temperature not being high enough to allow the excluded carbon to unite into a more or less continuous mesh, but that it is kept in very minute, separated aggregates, thus leaving the decarbonized iron crystals in immediate contact, as they are in ingot metal (steel). The advantage of this process lies in getting the final forms run from a perfectly fluid iron, at a comparatively low temperature thus obtaining smooth, full, and solid castings, which can then be "decarbonized" (chemically speaking), without allowing the excluded carbon to form in a graphitic matrix. The alternative is to melt and cast direct the decarbonized iron (steel), thus making what are known as *steel castings*. This requires a very much higher melting temperature, and thus the metal is less fluid, and it is apt to contain gases, or to generate them in the mould from the excessively high temperature at which it must be poured. The effect is that steel castings are apt to be rough and unsightly on the exterior and more or less porous on the interior.

When cast iron cools slowly from the melted state the carbon, which is wholly in solution or in chemical combination, passes largely into the graphitic form (as shown in Fig. 72), this graphite forming a complete matrix, and producing the dark, leaden appearance of gray cast iron. In all large or thick castings the cooling is of necessity slow (except when purposely chilled at the surface), and hence such forms are not radically changed

* *Trans. Am. Soc. C. E.*, vol. xxxiv (1895), "Notes on Manufacture and Properties of Malleable Cast Iron," by H. R. Stanford, Assoc. M. Am. Soc C. E.

in their molecular composition by the annealing which constitutes the essential feature of the malleable process. Only small castings, therefore, are suited to this process, unless a very hard white cast iron is used, which does not change in cooling to gray iron when cast in thicker or heavier masses. It is essential to the successful working of the process that the original castings shall have the carbon wholly, or nearly so, in the combined form, which then changes to an ununited graphitic form when kept some five days at a bright cherry-red heat. The only essential characteristics of the packing material are, according to Mr. Stanford, such as prevent it from fusing or adhering to the castings, or from caking together in hard lumps, and that it should not be too expensive. The packing serves only to exclude the air and to hold the cast forms to their normal shapes, when heated and softened, except as to the decarbonizing action of iron-scale packing on the superficial portion of the forms so treated, as described below.

81. Method of Manufacture.—The original castings, which have hitherto been made only of charcoal pig iron, may as well be made from coke pig iron,* provided the sulphur be kept low, the essential requirement being that the castings shall show all white iron (all carbon in the chemically combined form). These are then packed carefully in cast-iron annealing-pots, about 18 inches by 24 inches in cross-section, and four feet high, made in three sections for convenience of packing, the sections fitting together with bell and spigot ends. The castings are so placed in the pots that such settlement as occurs in the oven will not deform them. They are surrounded by the packing material, which is usually a decarbonizing agent for their outside portions, but which serves mainly only as a suitable bedding or packing for large cross-sections. When large forms are to be treated, they are placed and covered in the oven, without the use of annealing-pots. The iron pots waste away rapidly by oxidation, being able to serve only about five heats, of five days each.

The annealing-oven may be any suitable oven in which the temperature may be kept nearly constant, and uniform over its entire bed. This requires that a portion of the combustion shall be completed in the oven itself. About five days are required to fully effect the change in the condition of the carbon, after which the furnace is allowed to cool down, the first 24 hours with closed doors. The cost of the treatment is from one-fourth to one-half cent a pound

The effect of sulphur in the cast iron is to greatly delay the change in the carbon state, thus largely preventing it, for the ordinary periods of treatment or requiring a greatly extended period of annealing to fully accomplish it. Thus iron containing 0.04 per cent sulphur will anneal in three and one-half days, while iron in the same sizes containing 0.20 per cent sulphur requires about nine days. Hence if coke iron is used the sulphur ingredient must be looked to

* On the authority of Mr. Stanford. See paper quoted above.

Clean, heavy-forge iron-scale seems to be the best material to use for packing the castings in the annealing-pots, and these, being composed of iron oxide, having a strong affinity for carbon, do extract a large part of the carbon from the exterior $\frac{1}{16}$ inch of the surface of all castings so treated, leaving a bright-colored envelope (on the fracture), containing very little graphitic carbon. This skin is very much stronger than the interior, as shown by Fig. 59, from which it appears that the removal of this portion reduces the strength per square inch by 25 per cent. This argues that *the outer skin is more than twice as strong as the interior.** It is necessary to establish this fact by further experiment before accepting it as a general truth.

82. Mechanical Properties of Malleable Iron.—From what has been said it is evident that "malleable iron" is an extremely various product, depending on the materials used in the cast, and also on the treatment. The following table is taken from Mr. Stanford's paper, which shows what may be accomplished by this process. The test specimens were $\frac{13}{16}$ inch in diameter, and were tested without dressing down either on the gripped ends† or on the reduced portion. The small reduction in total carbon (which all occurred in the outer portions) and the change from combined to graphitic carbon are here shown conclusively. The average tensile strength of 49,800 lbs. per square inch is probably about twice that of the original castings, while the average elongation of 6.6 per cent indicates a very considerable ductility. This elongation of $\frac{1}{16}$ inch to the inch, together with an assumed corresponding compression, makes it apparent that small sections would submit to a considerable amount of bending distortion and other kinds of abuse before breaking. In other words, the iron is now twice as strong to resist a static load, and probably many hundred times as strong to resist the force of shocks or blows, as was the original cast iron.

The elastic limit in compression is very low, but the compressive deformation may be very great.

The relative strength and ductility acquired under varying periods of annealing, from 3 to 9 days, is shown in Fig. 59, where the averages of all the results given by Mr. Stanford are plotted, after being reduced to a common standard of reference. The few tests on turned-down specimens are also plotted, but these are too few to give either very accordant or very trustworthy results.

In Fig. 60 are shown the results of tension tests on malleable cast iron of $\frac{1}{4}$ inch and $\frac{7}{16}$ inch in thickness, and also of $\frac{1}{4}$-inch plates which had been welded together. These last show a greater strength than the unwelded bars.

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* Due allowance being made for the relative portions of each.

† These parts should have been dressed in order to prevent any want of symmetry in the applied forces, or forms like that shown for cast iron in Chapter XV could have been used. Some of the discrepancies in these results are due doubtless to the rough surfaces in the grips.

TABLE VI.—MALLEABLE CAST IRON.

CHEMICAL COMPOSITION (UNANNEALED AND ANNEALED) AND PHYSICAL PROPERTIES.

										Tests of Annealed Specimens.				
Heat No.	Unannealed or Annealed.	Total Carbon.	Combined Carbon.	Graphitic Carbon.	Mn.	Si.	P.	S.	Loss of Carbon.	Maximum Strength per Sq. In.	Per Cent. Reduction.	Per Cent. Elongation.	Number of Bars Averaged.	Hours Annealed.
391	Before annealing	3.02	2.96	0.06	0.18	0.69	0.138	0.066	0.07	55,100	5.5	5.2	3	108
	After "	2.95	0.15	2.80	0.19	0.69	0.140	0.064						
395	Before "	3.09	2.99	0.10	0.19	0.60	0.133	0.060	0.11	43,800	6.1	6.3	3	108
	After "	2.98	0.53	2.45	0.19	0.62	0.130	0.061						
400	Before "	3.09	2.96	0.13	0.18	0.75	0.142	0.065	0.53	44,900	7.7	10.3	3	108
	After "	2.56	0.31	2.25	0.19	0.74	0.142	0.061						
406	Before "	3.07	2.77	0.30	0.19	0.65	0.163	0.064	0.16	47,000	7.2	6.2	3	108
	After "	2.91	0.08	2.83	0.20	0.64	0.164	0.064						
422	Before "	3.26	2.65	0.61	0.17	0.69	0.156	0.071	0.31	45,300	9.1	8.2	3	108
	After "	2.95	0.36	2.59	0.18	0.61	0.151	0.068						
433	Before "	2.85	2.72	0.13	0.18	0.74	0.161	0.073	0.08	64,500	1.3	2.8	3	108
	After "	2.77	0.72	2.05	0.18	0.72	0.162	0.071						
436	Before "	2.88	2.73	0.15	0.18	0.90	0.196	0.069	0.30	38,900	4.3	6.2	3	108
	After "	2.58	0.51	2.07	0.18	0.87	0.192	0.069						
445	Before "	2.97	2.75	0.22	0.18	0.77	0.148	0.073	0.31	69,100	2.6	4.0	2	108
	After "	2.66	0.31	2.35	0.19	0.76	0.145	0.075						
451	Before "	3.08	2.85	0.23	0.18	0.96	0.123	0.033	0.93	45,400	7.2	8.0	2	108
	After "	2.15	0.08	2.07	0.19	0.96	0.129	0.039						
458	Before "	3.08	2.82	0.26	0.18	0.74	0.151	0.036	1.05	56,700	8.4	8.2	2	108
	After "	2.03	0.28	1.75	0.18	0.71	0.150	0.037						
459	Before "	3.03	2.81	0.22	0.19	0.70	0.195	0.038	0.97	44,200	2.1	4.2	3	108
	After "	2.06	0.27	1.79	0.20	0.70	0.192	0.037						
469	Before "	3.09	3.00	0.09	0.20	0.77	0.127	0.022	0.28	51,600	7.7	7.0	3	108
	After "	2.86	0.32	2.54	0.19	0.75	0.129	0.023						
474	Before "	3.08	2.92	0.16	0.23	0.70	0.158	0.023	0.41	46,600	9.8	8.5	3	108
	After "	2.67	0.09	2.58	0.22	0.70	0.156	0.023						
495	Before "	2.84	2.62	0.22	0.31	0.63	0.182	0.014	0.26	48,100	8.6	8.7	3	108
	After "	2.58	0.06	2.52	0.32	0.66	0.178	0.018						
510	Before "	3.26	3.23	0.03	0.40	0.64	0.136	0.040	0.14	46,000	5.9	5.3	3	108
	After "	3.12	0.53	2.59	0.40	0.67	0.135	0.039						
Av.	Before annealing	3.04	2.85	0.19	0.21	0.73	0.154	0.050	0.38	49,810	6.23	6.61	42	108
	After "	2.66	0.31	2.35	0.21	0.72	0.153	0.050						

NOTE.—The above test-bars were all cylindrical in section and 1⅛ inch in diameter

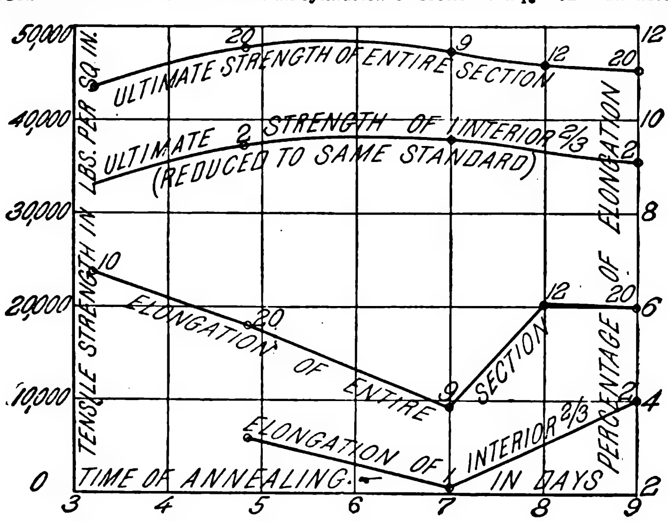


FIG. 59.—Tensile Strength and Per Cent of Elongation of Cylindrical Test-specimens of Malleable Cast Iron 1⅛ in. in Diameter. Figures show Number of Tests averaged for each Point Plotted. (Stanford, *Tr. Am. Soc. C. E.*, vol. XXXIV, 1895.)

In Plate IV the plain bar represents the original form of malleable iron from which all the other forms on the plate were worked. In one case it is

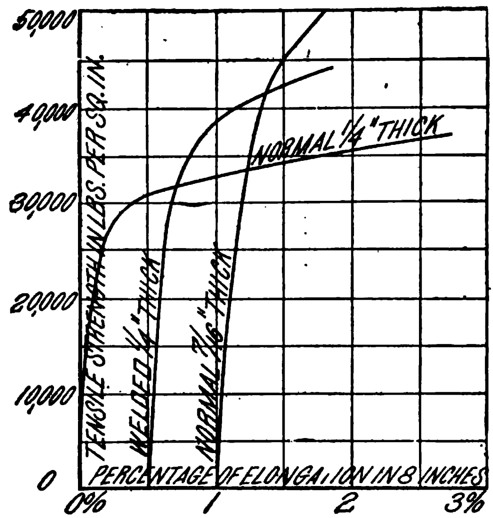
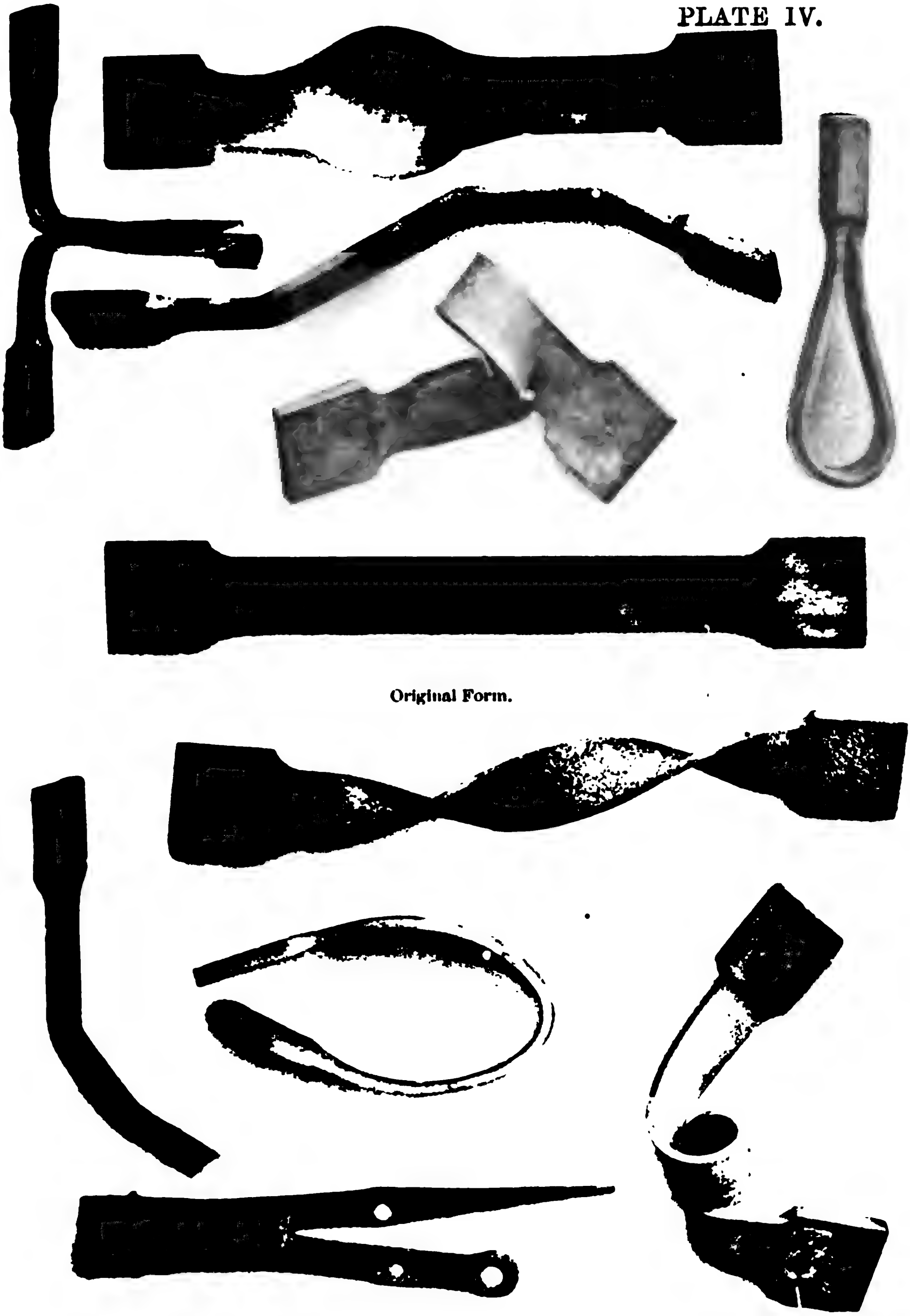


FIG. 60.—Tension Tests of Malleable Cast Iron. Each Curve the Mean of Two or Three Tests. (*Berlin Testing Laboratory*, 1886.)

folded over and the ends are welded together, while in another it has been forged like wrought iron. All the other forms were bent cold.

PLATE IV.



Examples of Cold-bending, Forging, and Welding of Malleable Cast-iron Specimens, all being Originally like the Undeformed One in the Centre of the Plate.

(Berlin Testing Laboratory Communications, vol. iv, Pl. III.)

CHAPTER VIII.

WROUGHT IRON.

83. Definition.—Wrought iron may be defined as nearly pure iron intermingled with more or less slag. As will appear from a study of the methods of production to be described, the iron is formed in a bath of melted slag (somewhat as butter is formed in churning), and when it aggregates into a pasty mass and is removed from the furnace, to be squeezed and rolled, some of this slag remains intimately associated with the iron. This gives to wrought iron a fibrous appearance (see Fig. 322) not found in any other metal.* In the most carefully made iron this fibrous appearance is uniform throughout the entire cross-section. It is not uncommon, however, especially in the cheaper grades of wrought iron, to find it largely and coarsely crystalline. Wrought iron melts only at a very high temperature, and assumes a perfectly plastic state through a considerable range of temperature below this melting heat, in which condition it is easily and perfectly welded. It is more or less ductile when cold, and will not harden when heated and quenched in water.

The oldest methods of production of wrought iron were all direct processes, obtaining the malleable product at one operation, or directly from the ore. While some modern processes also proceed on this plan, practically all the wrought iron of to-day is made from pig iron and various kinds of scrap by a puddling process.

METHODS OF MANUFACTURE.

84. The Puddling Process Briefly Stated.†—"The ordinary puddling furnace, is a single-bedded reverberatory of simple construction, formed externally of cast-iron plates, tied together with wrought-iron rods, and provided with suitable openings in front for the fire-hole and the working-door, and lined internally with refractory fire-brick. The crown of the furnace is also of fire-brick, and is open to the air. The bottom of the furnace is composed of three cast-iron plates, which rest upon an iron frame. The grate of the furnace has wrought-iron fire-bars, and is large in propor-

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* To develop this fibrous appearance nick a bar of wrought iron on one side and bend it double, and if possible split it down like a stick of timber. See Fig. 322.

† The quoted paragraphs on wrought iron have mostly been taken from Turner's *Metallurgy of Iron*, Lippincott & Co., 1895.

tion to the bed or crucible part on account of the very high temperature required, particularly towards the end of the process. Each puddling furnace is provided with a separate flue, which is either connected to a simple rectangular stack, provided with an iron damper, or which passes into a boiler-flue so as to economize the waste heat of the furnace. A sectional view of such a furnace is shown in Fig. 61.

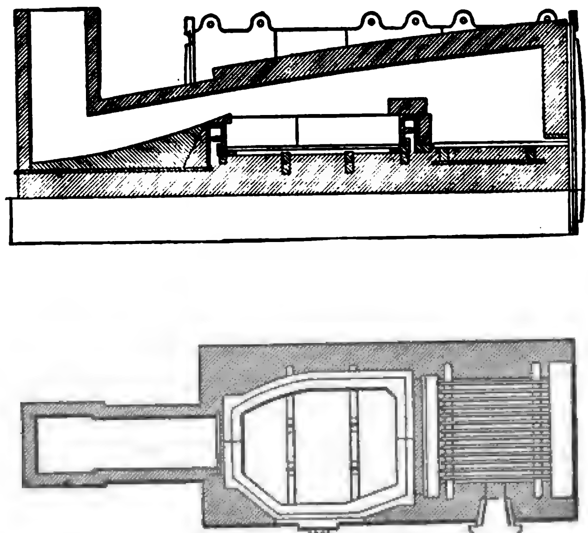

FIG. 61.—Plan and Section of a Simple Reverberatory Furnace.

"Two men are employed at each furnace, and are called the 'puddler' and the 'under-hand' respectively. The work is very laborious, while it entails no little skill if good results are to be obtained. Usually six heats are worked in a turn of twelve hours, but exceptionally seven heats are obtained.

"The furnace is first charged with a sufficiency of fluxing cinder or 'hammer slag' (oxide of iron) which has been squeezed out under the hammer from previous balls, and there is then introduced about 500 lbs. of good gray forge-iron. The door is closed and the charge is then heated to melt the iron, and the most favorable results are obtained when the iron and the cinder, charged as above described, become pasty, and melt down together. When the iron has thoroughly melted down and has become fluid, it is carefully watched until it has 'cleared,' and until a number of small blue jets of flame issue from the surface of the liquid. The damper is now 'put down,' or closed, so as to fill the furnace with a reducing (non-oxidizing) atmosphere, and to lower the temperature somewhat. In a short time the jets of blue flame almost cease, and the mixture of iron and cinder rises in the furnace to a height of some 8 or 10 inches, and during this stage constant stirring or 'rabbling' is necessary to prevent the iron settling on

the bottom of the furnace, and to assist the decarburization by bringing the (carburized) iron and cinder (iron oxide) into uniform and intimate contact. The whole mass should now be in motion, and bubbles of gas should rise and burn with a blue flame, tinged more or less with yellow, at the surface. When the 'boil' is thus in full progress, or 'well on,' the damper may be raised somewhat, and the iron will soon be observed to 'come to nature' or to separate from the cinder. The first sign of this is the appearance of small bright spots on the surface of the cinder, which alternately appear and disappear. The cinder now gradually sinks, and leaves the iron as an irregular mass, not unlike the small globules or grains of butter produced by the churn; and as in good butter-making so in good puddling, the grains should be small and uniform throughout the mass. The temperature should now be raised to the highest point so that the iron may be at a welding heat; the puddler after first lifting the metal and turning it over, by inserting a bar underneath in order to prevent the bottom becoming colder than the top, and breaking it up, proceeds to collect it into balls, which are taken to the hammer."

85. Oxidation in Puddling.—" The following remarks on the oxidation of cast iron under different conditions, will explain the differences between the old and newer processes of puddling.

" It is usual to speak of atmospheric air as oxidizing and removing the impurities present in cast iron, but if a globule of cast iron be melted in the air, and then exposed to a blast of air or oxygen, it will be observed that the impurities are not the only substances that are oxidized. It is true that under very special conditions either the carbon or the silicon may be separately oxidized. But on performing the experiment above indicated it will be found that the iron itself is oxidized in about the same relative proportion as the other elements, and the result is that practically a layer of impure magnetic oxide of iron is formed outside the globule, while the portion of metal that is left is of nearly the same composition as the original iron. If the cinder be allowed to run away as rapidly as it is formed, ultimately the whole of the iron would be converted into magnetic oxide, and the last particles of cast iron so removed would have nearly the same composition as the original metal. In this case oxidation has taken place, but no purification has resulted.

" If, now, the same experiment be tried, but the fluid oxide be allowed to remain, and to cover the fused metal, the oxidation of the iron will proceed very little further; a reducing action will then be commenced whereby the silicon, carbon, and other easily oxidizable elements will be removed, but at the same time a corresponding weight of iron will be returned to the globule from the surrounding slag.

" If, thirdly, a globule of cast iron be covered with magnetic oxide of iron to protect it from the air and to supply the necessary cinder, and it be then strongly heated, it will be found that the globule has not lost in weight, but has become distinctly heavier during the process. It is scarcely neces-

sary to say that the waste which takes place during reheating or remelting, corresponds to the first condition above given. The oxide runs away as it is formed, and this is an example of waste of iron pure and simple. The only redeeming feature is that sometimes the oxide produced may be of value for other purposes. The early open-hearth processes for producing wrought iron in fineries, and the original method of puddling, resemble the second case, for part of the iron is wasted to produce the cinder needed to remove the impurities from the remainder of the metal. The larger the proportion of these impurities the greater will be the loss of iron necessary to make the required cinder, and for this reason a comparatively pure iron is needed, in order to obtain the least waste, while at best the waste is comparatively great. A deficiency of fluid cinder in the early stages of ordinary puddling, or pig 'boiling,' has an exactly similar effect, and leads to waste for the same reasons.

"In the modern method of working, on the other hand, the object is to imitate the conditions of the third case previously supposed. Oxide of iron can be bought much more cheaply than it can be made from pig iron, and besides the oxidation of pig iron requires the expenditure of time and fuel. Oxide of iron is, therefore, supplied in its cheapest and most readily available form, and as much of this oxide as possible is reduced and converted into wrought iron. To do this it is necessary that the iron and fluid oxide should be brought into actual and frequent contact, and so perfect fluidity and constant rabbling are needed. There is, of course, a practical limit to the amount of carbon which can be present, due to the fact that cast iron cannot take up more than a certain amount, say 4 per cent, of this element. There is also a practical limit in the case of both silicon and phosphorus; the first being regulated by the increased consumption of time and fettling with excess of silicon, and the second being determined by the inferior quality of iron produced, with large proportions of phosphorus. But within these practicable limits it is advantageous to reduce as much of the oxides of iron supplied as possible."

86. Details of the Puddling Process.—"The working heat of puddled iron may be conveniently divided into four stages, which will be separately described, namely:

"(1) *Melting-down stage*, lasting about half an hour, by the end of which most of the silicon and manganese and a considerable proportion of phosphorus have been removed.

"(2) *Quiet fusion or 'clearing' stage*, lasting about ten minutes, during which the rest of the silicon and manganese and a further quantity of phosphorus are removed.

"(3) *The boil*, which lasts nearly half an hour, during which the greater part of the carbon is eliminated, together with a further quantity of phosphorus.

"(4) *Balling-up stage*, which occupies some twenty minutes, and by

which time the purification, except as regards the removal of slag, has practically ceased.

"1. The furnace having been suitably prepared, and hot from a previous heat, the pig iron is charged as before described; the door is then closed, and the working opening in the bottom of the door covered with an iron plate and rendered as far as possible air-tight by means of a little fine cinder thrown with the shovel. The fire is also made up, and heating proceeds for some twenty minutes, by which time the top of the pig iron is red-hot, and the flux begins to soften. The pigs are now turned so as to heat them more uniformly and the door is again closed; in a few minutes the iron begins to melt, and if carefully watched may be seen to trickle down into the cinder in drops. The workman now introduces an iron rod, stirs up the mass, and brings up any pieces of iron which have not completely melted, and which might otherwise remain covered and take longer to melt. When the whole is thoroughly fluid and well mixed the melting-down stage is finished.

"2. One of the workmen, generally the under-hand, now introduces a bar which is bent at the end at right angles, and so acts as a scraper or stirrer, and the whole charge is well stirred and exposed to the action of the fettling and cinder, and also to some extent to the oxidizing influence of the air. The temperature is maintained as high as possible during this stage.

"The iron is thus thoroughly 'cleared' or purified from silicon, the point at which clearing is completed being judged by the appearance of the charge, and upon the skill of the workman at this stage much of the subsequent success depends.

"3. When the metal has cleared, and is in a state of tranquil fusion, the next point is to bring on the 'boil.' The puddler, therefore, diminishes the draught, or 'puts his damper down,' so as to fill the furnace with a smoky flame and lower the temperature. In some cases also the door is opened and water thrown in at this stage, as this promotes rapid cooling and supplies oxygen at the same time. The (carbonized) metal being thus somewhat thickened, and being vigorously stirred during the whole time, becomes intimately mixed with the (iron oxide) cinder; the carbon is thus oxidized, producing carbon monoxide, which burns in blue flames as the bubbles of gas rise and burst. These flames are sometimes called 'sulphur' or 'puddler's candles,' on account of their pale blue color. The charge thus swells up and rises some six inches in the furnace (like boiling molasses), and as the heat increases and the damper is opened somewhat, a quantity of red-hot slag flows over the fire-plate (at the door) into a cast-iron slag-wagon placed ready to receive it. The violence of the action now gradually diminishes, the iron 'comes to nature,' and the charge settles in the furnace; the less fusible wrought iron is in the form of a porous cake, and the residue of slag collects chiefly underneath.

"4. In the fourth, and last, stage the puddler has to manipulate the iron into convenient forms for subsequent treatment. For this purpose the

cake of metal is broken up by inserting a bar underneath, and is worked at a welding heat into one uniform mass or ball. This is now divided into about six balls, of approximately equal size, each of which weighs about 80 lbs., and these are in turn withdrawn from the furnace and taken to the hammer, where the slag is to a great extent expelled, and a bloom of iron is obtained. This is rolled, without reheating, into 'puddled bar,' which is the name given to the crude wrought iron produced as above described."

87. Production of Muck Bars.—" The balls of crude wrought iron, having been produced in the puddling-furnace as before described, have now to be compressed to expel the slag and render the material more uniform in character; they are afterwards rolled into bars, which receive the name of 'muck bars.' For compressing the iron various forms of hammers or squeezers are used, while for the production of bars, grooved rolls, as introduced by Cort in 1783, are generally employed, though in a few exceptional cases, where water-power is available, bars are still produced by the hammer or 'battery,' as in ancient times.

" Various forms of squeezers have been introduced from time to time, chiefly with the object of preventing the jar or shock due to the action of the hammer, though such appliances have not met with very general application. The more usual forms may be conveniently divided into two classes:

" (1) Those in which compression is produced by means of a lever, as in the 'alligator' or 'crocodile' squeezers, which are so called by the workmen from the resemblance between the motion of this class of squeezer and that of the mouths of the animals above mentioned.

" (2) Those in which a revolving cam is employed.

" Though squeezers appear at first sight to have many advantages over hammers, particularly on account of their even and quiet action, they do not seem to have grown in general favor in recent years, it being stated that the iron worked in squeezers is less uniform in character, and that the slag is not so completely expelled by squeezers as with hammers.

" Steam hammers are used for shingling puddled balls in almost all modern works, and are now always double-acting. The hammer-block in this instance weighs about ten tons, and is heavier than is generally employed in forges, though lighter than is usual for manipulating large masses of steel. Forge-hammers seldom exceed three tons in weight, while steam-hammers for forgings of the largest size weigh 50 tons and upwards.

" The iron, having been thus compressed and consolidated by some form of hammer or squeezer, and a considerable portion of the slag expelled, is now taken while still hot to the puddle-rolls, where it is converted into bars. The bars are allowed to cool, and are afterwards cut up with shears into suitable lengths; these are then made into bundles, or 'piles,' of the required weight and size. When a specially smooth surface is required, as in the production of sheet iron, it is usual to make the top and bottom of each pile of 'scrap bars'; these are made by reheating the crop ends of finished bars or other good wrought-iron scrap, and are therefore more uniform in char-

acter, and possess a smoother and cleaner surface than ordinary puddled iron."

88. Reheating the Muck Bars.—" The puddled iron having been prepared as before described, is now taken from the forge to the other part of the works, which is known as the 'mill.' This is usually covered with a tolerably lofty roof, but is open at the sides; it contains reverberatory furnaces for heating the piles of puddled iron, and also rolls of various sizes, with the necessary engine and connections required for producing the various 'sections' of finished iron. A steam-hammer is also provided if forgings are produced, but otherwise this is not required.

" The temperature employed in mill-furnaces is a white heat, and sufficiently high to cause the metal to weld together when it is passed through the rolls, to which it is taken from the furnace."

89. Rolls.—" The rolls used in iron-works are classified according to their shape and the method adopted in their production. They are generally made from a strong close-grained cast iron, usually that obtained from a blast-furnace, in which cold blast is employed. Occasionally steel rolls are used, and these appear to be somewhat growing in favor in recent years.

" Rolls may be classified according to their shape into—

" (1) *Flat or plain rolls*, which are used for rolling sheets or plates.

" (2) *Grooved rolls*, which are required for the production of bars, rods, angle and channel iron.

" According to their method of production rolls are classified as—

" (1) *Grain rolls*, which are produced in moulds of green or dry sand, and in which the surface of the roll after turning down shows the ordinary grain of the cast iron from which it is made. These are used for all roughing purposes and for sections, and in other cases if the metal is finished hot.

" (2) *Chilled rolls*, which are produced in cast-iron moulds or chills. They, therefore, have a hard white surface of chilled iron, which varies in thickness from about $\frac{1}{4}$ to $\frac{3}{4}$ of an inch, according to the size of the casting and the class of work for which it is intended. Rolls of this kind are more costly, and are employed for the production of sheets, plates, or strip, or in other cases where specially fine surfaces are required."

Small rolls tend more to elongate than to spread the materials, while large rolls tend both to elongate and to spread the metal.

90. Effect of Repeated Reheating of Iron.—"As it is well recognized that puddled iron is much improved in quality by being cut up, piled, reheated, and rolled or hammered, and that the iron is further improved by repeating the operation, it might be assumed that by continuing this process the properties of the metal might be again and again further improved. In practice, however, this is not found to be the case, and it is only in special cases that it is advantageous to reheat puddled iron more than twice. It has been shown by experiments, in which puddled bar was reheated and rolled as many as twelve times, that after about six workings the metal began to seriously deteriorate, and even in the earlier workings, after the third no

corresponding advantage was obtained for the fuel and labor expended and the waste incurred. The results obtained were as follows (*Useful Metals*, p. 318):

	Tensile Strength in Pounds per Square Inch.
Original puddled bar	43,900
2d working	52,860
3d "	59,580
4th "	59,580
5th "	57,340
6th "	61,820
7th "	59,580
8th "	57,340
9th "	57,340
10th "	54,100
11th "	51,970
12th "	43,900

"If it be assumed that the result in the fifth heating was accidentally low, it will be seen that all the other tests follow in a regular succession, the maximum tensile strength being obtained with the sixth working. Probably with iron of different composition or character the maximum would be reached at a different point, but in all cases the gradual original improvement and subsequent deterioration would be observed. When the metal passes into the hands of the smith it is found that if it has been worked during its previous preparation so as to bring it to its best condition, it has a tendency to 'go back' in forging; while, on the other hand, if the iron has not been unduly worked, it improves when properly smithed. For this reason also it is not advantageous to often reheat and work iron during the process of manufacture, and 'best,' 'best best,' or 'treble best' irons are obtained, not by frequent heatings, as is sometimes stated, but by the careful selection of all the materials employed, and by systematic and frequent tests of the iron during the various stages of manufacture."

91. Sections of Finished Iron.—"The shape into which finished iron is rolled varies according to the purposes for which it is designed, the chief divisions being plates, sheets, strips, bars, angle-irons, and rails, the last being relatively of much less importance than formerly. Among the more usual shapes or 'sections' may be mentioned the following: bars, including round, half-round, square, flat, round-edged flats, oval, octagon, together with levelled and bulb iron, and rods; tee (or T-shaped) iron, tee with round top or edges; angle- (or L-shaped) iron, angle-iron with unequal sides or round back; channel-iron, I iron, Z iron; rails, including single-headed, double-headed, and flange; and horseshoe-iron, which is rolled single-grooved, double-grooved, or concave. Numerous other forms are also required from time to time for various purposes; so that the number of rolls which have to

be kept in stock at a large works with a general trade is very great, not unfrequently amounting to hundreds. As each pair of rolls is generally only capable of finishing one section of iron, the cost of the supply and maintenance of rolls forms a considerable item of the expenditure of an ironworks."

92. Imperfections in Finished Iron.—"The three chief varieties of imperfection in the appearance of finished iron are rough edges, spilly places, and blisters.

"(*a*) *Rough edges*, when not due to imperfections in the rolls or to careless working, are a sign of red-shortness, and are particularly noticeable in flat bars or strips. Red-shortness may be due to an excess of carbon, or to the presence of sulphur, particularly if copper is also present. Usually, however, if iron has been properly puddled, practically the whole of the sulphur is eliminated, and the red-short condition is due to the 'dryness' of the iron. Iron is said to be dry when it is deficient in fusible or welding cinder, which may be readily squeezed out from between the particles when the iron is worked, and so enable clean surfaces to be brought together to form a good weld. A thick dry cinder, on the other hand, leads to red-shortness, and a piece of brick or other foreign matter which crushes up in the rolls to form a dry powder acts in the same manner.

"(*b*) *Spilly places* are spongy or irregularly spotted parts which are not unfrequently noticed in sheets, and which are occasionally met with in all kinds of wrought iron. They are generally due to imperfect puddling, whereby one part of the iron, when 'coming to nature,' has been oxidized more than another. If the heat has been thoroughly well worked and the iron uniformly mixed, spilly places are seldom observed.

"(*c*) *Blisters* are not unfrequently met with in sheets, and lead to considerable loss and inconvenience. They are much less common in steel sheets than in iron, and some experiments conducted in 1893 led the author to attribute the formation of blisters to a reaction between carbon and oxide of iron in wrought iron of inferior quality. This view is in accordance with the experiments of A. Friedmann, who collected and analyzed the gas contained in a number of blisters. This gas was found to contain over 70 per cent of carbon monoxide, the remainder being chiefly carbon dioxide, with some nitrogen and hydrogen. Inside the blisters a quantity of scaly matter is found, which Friedmann states to consist of about two thirds silica and nearly one third iron aluminate ($FeAlO_2$), together with small quantities of other oxides." *

MECHANICAL PROPERTIES OF WROUGHT IRON.

93. Crystalline Fracture.—As explained in Art. 84, the fibrous appearance of wrought iron when nicked and bent with a splitting action, similar to that of a piece of timber treated in like manner, is due to the presence of

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* *Inst. Journ.*, 1885, vol II. p. 645.

the foreign matter which formed the slag or bath from which the puddled ball was taken. Ordinarily when wrought iron is broken in tension in a testing-machine the fracture appears to be wholly fibrous, somewhat like that of soft steel, but with a darker and more ragged appearance. If a wrought-iron bar be nicked and broken by bending, it will usually show a fibrous appearance, whereas steel so treated will always show a crystalline fracture. Occasionally, however, a part of all of the fracture of a test specimen of wrought iron, whether broken in tension or by nicking and cross-bending, will have a coarsely crystalline fracture. It is very common, also, to find such a fracture when wrought iron breaks in service, as in the case of car and wagon axles, steam-engine cranks and pins, etc. In such cases as these it has been common to ascribe the failure to the crystallized condition of the iron and to assume that the iron had changed to this condition in service. This is called the theory of the cold crystallization of wrought iron. Those who believe in it usually ascribe the change to a vibratory action. Whether or not wrought iron ever does crystallize in service in this or in any other manner has been a disputed question for the last half-century. It has, however, remained a theory the truth of which has never been established by actual experiment, and it is now one which seems to have no scientific adherents. It has, however, become so thoroughly fixed in the minds of the less educated users of iron and steel that it is met with on every hand, and this action is stated to be a fact with the most positive assurance by nearly all mechanics and is commonly believed by the public generally. The views of the author of this work on this subject may be summarized as follows:

I. The normal molecular arrangement of wrought iron is crystalline, but the thorough admixture of the inert slag in a well-worked product prevents these crystals from forming in visible sizes. The ordinary fibrous fracture, therefore, exhibits rather a lateral view of these finely crystallized threads, thus causing this to present a fibrous appearance.

II. When any portion of the puddled ball as removed from the furnace is not intermixed with foreign matter, as may be the case from overheating and melting of some portion of the puddled ball, or from the inclusion in the mass of some unreduced melted cast iron, these portions being really of the nature of ingot metal or steel, rather than of wrought iron, then these masses of iron, free from foreign matter, when somewhat cooled and rolled into a bar, will form in that bar a part of the cross-section which will be able to crystallize on a slow cooling in large-sized crystals, so as to be clearly visible to the naked eye. When such melted portions are due to overheating of the puddled ball the iron is said to be burnt, but a too-rapid hurrying of the boiling process under a low heat will also enable some of the unreduced cast iron to be removed from the furnace in this way with a similar result.

III. With the ordinary and more inferior grades of wrought iron now on the American market, it is very common to find large portions of the cross-section of test-bars showing a crystalline appearance, even for tension-test specimens of standard form. Much more, therefore, are such irons likely to

have this appearance when nicked and broken across, or when nicked and pulled in tension.

IV. All wrought iron when broken with *extreme suddenness* will show a crystalline fracture. This is because time is not given for the drawing out of the section, rupture occurring directly across the fibres, so that the fracture shows only the end view of the same.

V. When a bar is nicked with a sharp chisel, or grooved in a lathe with a sharp-pointed tool, and broken across, rupture begins at one side without any elongation of the fibres, and extends from fibre to fibre across the section in such a way as to produce a result similar to that caused by an instantaneous rupture cited in IV. In this way wrought iron will often show a crystalline or granular fracture, when under the ordinary tensile test it would be wholly fibrous. All steel or ingot metal will always show a crystalline fracture when treated in this manner, although for all the soft and medium grades of steel the fracture is always fibrous or silky when broken in tension, with the usual accompanying elongation and contraction.

VI. Much of the so-called wrought iron on the market to-day consists simply of rolled fagots of "scrap-iron," a large portion of which is really scrap-steel. As these are heated only to a welding heat, and then rolled into merchant bar, there is no real mixing of the metals, and the several components form so many separate portions of the cross-section of the final rolled forms. The crystallized steely areas found in the fractures of most wrought irons of the common grades to-day can be largely traced to this source. Wrought-iron railway-axles and other large forms are usually made up in this way.

VII. When wrought iron breaks in service, therefore, and shows a coarsely crystalline fracture, it does not prove that crystallization has occurred in service. It proves only that this iron had such a structure originally. If, however, the rupture occurs in practice in a suddenly contracted area, as in a screw-thread or in a sharp angle, or if it has been produced with extreme suddenness, as in case of an explosion or shock of any kind, if the appearance of the fracture is finely crystalline or granular, this appearance may be wholly due to the method of failure. This is shown by the fact that if a specimen be cut from the adjoining metal and tested in tension with the standard form of specimen, it might show a wholly fibrous fracture. In such cases, therefore, the crystalline appearance of the fracture is due to the particular conditions as to shape of specimen and suddenness of rupture and not to any molecular change which has taken place in the iron.*

* From the *Report of U. S. Watertown Arsenal Tests of Metals* for 1890, in which are recorded many tests of specimens cut from the journals of old railway-axles, the following note is taken:

"Axles have been examined which have had long-continued service, the journals of which showed incipient cracks, indicating that rupture had begun and that further use must result in complete rupture. It is a remarkable fact that the tests of the metal of these journals near these cracks showed no loss of strength or ductility. No indications

94. The Welding of Wrought Iron.—It is a peculiar property of wrought iron that it remains in a plastic condition throughout a considerable range of temperature. If two pieces of wrought iron could be reduced to this plastic state (a white heat) with perfectly clean surfaces, and pressed together firmly and allowed to cool, the union would be so perfect as to be practically as strong as any other portion of the material. The great difficulty in successful welding lies in the fact that when the iron is heated in the presence of oxygen the surface is oxidized, and this oxide of iron, being quite fusible at this temperature, forms a complete coating of slag over the entire surface. When two such surfaces are brought together, therefore, each being entirely covered with melted iron oxide, which is practically a foreign substance, the union effected is necessarily imperfect. The degree of imperfection depends on the amount of this melted slag which succeeds in remaining in the joint. In order to remove this liquid slag as much as possible, *the two surfaces should be convex to each other when they are brought together.* That is to say, they should first come in contact along the central portion of the weld area, so that the hammering or the pressure by which the weld is effected will, as perfectly as possible, squeeze out this liquid slag from the joint, thus allowing the plastic iron surfaces to come into immediate and actual union. If any portion of this melted oxide remains in the joint, it entirely prevents a union of the surfaces over such area as it occupies, and to that extent weakens the joint. As it is impracticable to heat the surfaces to be welded or even to join them in a vacuum, or away from the oxygen of the air, it is impossible to avoid entirely the presence of the melted oxide of iron in welding operations. With intelligence and care, however, in the performance of the work, nearly all this oxide can be removed in the act of welding, and a practically perfect union effected. To assist in removing this melted oxide, borax is commonly used. This being a perfect solvent of the oxide, the whole is changed to a thin liquid, which is the more perfectly squeezed out of the joint in the welding process. In this way steel may be welded which would not unite without it. When the parts to be joined are heated in an ordinary forge the blast of air causes an excessive oxidation of the surfaces, and thus gives rise to large quantities of the melted slag. By maintaining a thick fire most of the oxygen has been consumed to CO or CO_2 before

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of a tendency to crystallize were discovered, and inasmuch as the metal has gone through all the phases of deterioration up to the limit of actual rupture without showing a crystalline tendency, it is thought this demonstrates and proves that this material is incapable of cold crystallization when exposed to the conditions of service."

In one instance one of the old cracks which had developed at the inner shoulder of the journal reached to a depth of 0.02 inch into the side of the test specimen, and yet the specimen broke two inches from this section. After rupture the end of the specimen (1¼ in. diam.) containing this crack was bent cold 83 degrees with "this crack at the middle of the bend on the tension side, which opened the crack in width and also developed numerous cracks in this vicinity," but without rupture. All the tests showed fibrous fractures.

reaching the iron, and hence less oxide is formed. If the parts were heated in a reverberatory furnace or in a "muffler," raised to a sufficient temperature and kept out of the way of air-currents, a much less amount of this slag would be formed, and the welding would be more readily performed. One of the advantages of electric welding lies in the fact that no air-current is employed, and by having the parts in contact during the time they are being heated, the air is largely excluded from the welding surfaces, and hence little or no oxide is formed there to prevent a perfect union. It is largely for this reason that electric welding may be more perfect than hand welding.

In view of the inherent difficulties described above, it might well be anticipated that welded joints are necessarily very unreliable even when done with more than ordinary care. Many tests of the strength of welded joints have shown that this strength may be anywhere from 30 to 100 per cent of the strength of the parts which have been joined, and in the hands of careless or incompetent workmen the strength of a welded joint may be almost zero. With the most careful work, however, that is found to be practicable in the best forging practice, the average strength of hand-welded joints has been found by Kirkaldy * to be in the case of round iron tie-bars from 1⅛ to 3⅛ inches in diameter, but 60 per cent of the average strength of the bars. In the case of flat plates from 2½ to 6 inches in width and from ¼ inch to 1 inch in thickness, the average strength of the welds was 71 per cent of the strength of the plates. In the case of chain-link welds from ½ to 2½ inches in diameter, the average of 216 tests showed an average strength of the welded joints of 83 per cent of the strength of the iron rods. In the case of a welded chain, where the strength of the chain is only that of its weakest link, it would not be safe to rely on a strength of joint greater than 50 per cent of the strength of the iron from which the chain has been made.

In 1885 Professor Bauschinger undertook an elaborate series of experiments to determine the relative welding qualities of soft steel and wrought iron, also the relative efficiencies of forging by hand and under a steam-hammer. The results of his experiments have been condensed in the following tables. These results show a strength of welded soft-steel bars equal to 89.2 per cent of the strength of the original material, while the efficiency of the welds of the wrought-iron bars was 95.6 per cent. The relative value of hand and power forging is indicated in the second table, where it is shown that the hand forging gave an efficiency of 84 per cent, while the steam forging gave an efficiency of 97.2 per cent, on the soft-steel bars, while on wrought iron these were 87.9 per cent and 91.0 per cent respectively.

These tests were made under the most favorable conditions, and they probably represent the highest attainable efficiency in welding on both kinds of materials. These results should, therefore, not be taken as representing average results in practice, but rather as an ideal which may possibly be

* Kirkaldy's *System of Mechanical Testing*, London, 1891, report KK.

reached with the greatest care. It will be noticed that the fourth soft-steel specimen gave an efficiency of 99.6 per cent, the break occurring entirely outside of the weld; while the sixth set of specimens of soft steel gave an efficiency of but 57.3 per cent, the break occurring in the weld. Another specimen of soft steel would not weld at all.

TABLE VII.—BAUSCHINGER'S TESTS OF THE STRENGTH OF WELDS WITH LOW-CARBON STEEL (INGOT IRON) AND WROUGHT IRON.

(Each line of "welded" results contains the mean of two tests.)

SOFT STEEL OR INGOT IRON

Dimensions of Original Cross-section in Inches.	Cross-section of Test-bar. Dimensions.	Area.	Condition of Bar.	Method of Welding.	Yield-point.	Tensile Strength.	Ratio: Welded / Original	Percentage of Elongation for 10 Inches.	Percentage of Reduction of Area.	Remarks.
3.15×1.18	2.19×0.72	1.58	Orig.	One heat	39,100	61,860		31.3	59	
3.15×1.18	2.26×0.71	1.60	Welded	Steam hammer	38,390	58,940	95.3	12.8	14	Both broke in weld
3.15×0.98	2.17×0.72	1.56	Orig.	One heat	35,260	59,580		28.2	56	
3.15×0.98	2.11×0.71	1.50	Welded	Steam hammer	38,190	62,560	105.0	12.5	13	Both broke in weld
1.77×0.87	0.98×0.72	0.71	Orig.	One heat	42,660	69,240		23.1	42	
1.77×0.87	1.07×0.70	0.75	Welded	Steam hammer	44,700	70,740	102.2	13.1	15	Broke in weld
1.34×0.59	0.58×0.59	0.34	Orig.	Two heats	43,650	69,100		24.7	52	
1.34×0.59	0.64×0.54	0.35	Welded	Hand forging	38,390	68,820	99.6	17.2	48	Broke outside of weld
1.26×0.55	0.55×0.55	0.30	Orig.	Two heats	42,660	65,400		29.8	65	
1.26×0.55	0.60×0.54	0.32	Welded	Hand forging	34,840	60,640	92.7	15.4	68	Broke outside of weld
1.18×1.18	$d = 0.70$	0.39	Orig.	Two heats	46,640	69,960		22.8	42	
1.18×1.18		0.39	Welded	Hand forging	36,970	40,100	57.3	0	0	Broke in weld
$d = 1.10$	$d = 0.70$	0.39	Orig.	Two heats	33,840	61,570		11.9	15	
$d = 1.06$	$d = 0.61$	0.33	Welded	Hand forging	35,550	45,790	74.4	0.9	6	Broke in weld
$d = 0.79$	$d = 0.44$	0.15	Orig.	Two heats	44,080	66,830		23.2	67	
$d = 0.79$	$d = 0.44$	0.15	Welded	Hand forging	39,100	58,230	87.1	8.7	17	Broke in weld
						Average =	89.2			

WROUGHT IRON.

Dimensions of Original Cross-section in Inches.	Cross-section of Test-bar. Dimensions.	Area.	Condition of Bar.	Method of Welding.	Yield-point.	Tensile Strength.	Ratio: Welded / Original	Percentage of Elongation for 10 Inches.	Percentage of Reduction of Area.	Remarks.
3.27×0.71	2.39×0.71	1.70	Orig.	Three heats	32,700	52,600		26.1	42	
3.27×0.71	2.36×0.59	1.39	Welded	Steam hammer	24,170	50,050	95.1	13.4	23	Broke in weld
2.56×1.06	1.64×0.72	1.18	Orig.	One heat	22,750	50,050		28.5	45	
2.56×1.06	1.65×0.69	1.14	Welded	Steam hammer	22,750	50,050	100.0	20.9	34	Broke in weld
1.65×0.47	0.84×0.47	0.40	Orig.	Two heats	27,730	51,190		11.4	18	
1.65×0.47	0.86×0.43	0.37	Welded	Hand forging	22,750	48,910	95.6	8.1	14	Broke in weld
1.34×0.63	0.58×0.64	0.37	Orig.	Two heats	28,440	56,880		24.3	42	
1.34×0.63	0.68×0.58	0.40	Welded	Hand forging	27,020	55,450	97.5	15.3	...	Broke in weld
1.02×1.02	$d = 0.59$	0.27	Orig.	Two heats	29,860	55,030		21.5	39	
1.02×1.02	$d = 0.59$	0.27	Welded	Hand forging	28,440	56,310	102.3	15.6	17	Broke near weld
$d = 1.02$	$d = 0.59$	0.27	Orig.	Two heats	29,860	61,000		18.3	36	
$d = 1.02$	$d = 0.59$	0.27	Welded	Hand forging	27,020	50,620	83.0	9.2	16	Broke in weld
						Average =	95.6			

TABLE VIII.—BAUSCHINGER'S TESTS OF THE RELATIVE VALUE OF HAND AND STEAM FORGING.

(Welding Low-carbon Steel and Wrought Iron.)

SOFT STEEL OR INGOT IRON.

Dimensions of Original Cross-section in Inches.	Cross-section of Test-bar. Dimensions.	Area.	Condition of Bar.	Method of Welding.	Yield-point.	Tensile Strength.	Ratio · $\frac{\text{Original}}{\text{Welded}}$.	Percentage of Elongation for 10 Inches.	Percentage of Reduction of Area.	Remarks.
1.26×0.47	0.81×0.41	0.33	Orig.		42,660	68,110		23.9	66	
1.26×0.47	0.82×0.36	0.30	Welded	Hand forging	47,300	68,250	100.2	15.7	66	Broke outside of weld
1.26×0.47	0.80×0.35	0.28	Welded	Steam hammer	47,490	71,100	104.5	20.9	62.5	Broke outside of weld
0.75×0.75	$d = 0.43$	0.14	Orig.		45,640	66,260		21.7	68	
0.75×0.75	$d = 0.43$	0.14	Welded	Hand forging	44,230	61,710	93.1	9.1	9	Broke in weld
0.75×0.75	$d = 0.43$	0.14	Welded	Steam hammer	39,810	65,550	98.9	13.0	36	Broke in weld
$d = 1.02$	$d = 0.59$	0.27	Orig.		42,230	62,700		22.9	63	
$d = 1.02$	$d = 0.58$	0.27	Welded	Hand forging	24,880	27,300	43.5	0.1	1	Broke in weld
$d = 1.02$	$d = 0.58$	0.27	Welded	Steam hammer	33,130	54,600	87.1	6.9	18	Broke in weld
$d = 0.91$	$d = 0.51$	0.21	Orig.		49,200	69,530		23.0	62	
$d = 0.91$	$d = 0.51$	0.21	Welded	Hand forging	44,930	68,960	99.2	15.8	28.5	Broke in weld
$d = 0.91$	$d = 0.51$	0.21	Welded	Steam hammer	44,930	68,400	98.4	14.8	20	Broke outside of weld

Average hand forging = 84.0
" steam " = 97.2

WROUGHT IRON.

Dimensions of Original Cross-section in Inches.	Cross-section of Test-bar. Dimensions.	Area.	Condition of Bar.	Method of Welding.	Yield-point.	Tensile Strength.	Ratio · $\frac{\text{Original}}{\text{Welded}}$.	Percentage of Elongation for 10 Inches.	Percentage of Reduction of Area.	Remarks.
$d = 1.02$	$d = 0.58$	0.27	Orig.		24,880	55,460		23.1	41	
$d = 1.02$	$d = 0.58$	0.27	Welded	Hand forging	24,880	48,770	87.9	9.9	10.5	Broke in weld
$d = 1.02$	$d = 0.58$	0.27	Welded	Steam hammer	27,020	50,480	91.0	12.7	35	Broke outside of weld

95. The Effect of Reduction in the Rolls on the Strength of Wrought Iron.*—Other things being equal, the strength of wrought iron will vary directly with the amount of reduction in the rolls from the size of the pile to that of the finished specimen. If it is desired to obtain an equal strength

TABLE IX.—EFFECT OF VARYING REDUCTION IN THE ROLLS ON THE STRENGTH OF WROUGHT IRON.

Diameter.	Length.	Elastic Limit.	Ultimate Strength.	Per Cent of Elongation.	Ratio of Elastic Limit to Ultimate Strength.
1	4.75	38,000	57,533	17.1	66.1
1⅛	6.25	34,600	54,850	21.65	62.9
1¼	7.50	32,600	53,350	23.5	60.8
1⅜	8.50	34,800	52,675	26.0	65.7
1½	9.75	33,100	52,400	25.3	63.3
1⅝	8.175	34,325	54,175	22.0	63.4
1¾	9.50	33,175	52,000	23.15	63.6
1⅞	9.75	31,875	50,325	23.2	63.8
2	10.50	31,800	49,725	22.3	63.9

* See also similar results on steel in Chap. XXV.

for different finished sizes, it is necessary to make these several sizes from the piles whose areas of cross-section bear a constant ratio to those of the finished sections. The following table gives average results of four series of tests on wrought iron on sizes from one inch to two inches in diameter.

As showing that a uniform reduction in the rolls may be made to produce iron of equal strength for these same sizes, the following table of results is given, the iron having been rolled and the tests of strength made expressly to establish this fact.*

TABLE X.—DIMENSIONS AND AREAS OF PILES, AREAS OF BARS IN PERCENTAGE OF AREAS OF PILES, TENSILE STRENGTH, ELASTIC LIMIT, ETC., OF NINE BARS.

Size of Bar.	Dimensions of Piles.	Area of Piles.	Area of Bars in Per Cent of Area of Piles.	Tensile Strength.	Elastic Limit.
Inches.	Inches.	Sq. In.	Per Cent.	Pounds.	Pounds.
2	8 × 10	80	3.92	50,763	33,258
1⅞	8 × 10	80	3.45	53,361	35,032
1¾	8 × 9	72	3.34	53,154	35,323
1⅝	8 × 8	64	3.24	53,329	33,520
1½	6 × 9	54	3.27	52,819	34,840
1⅜	6 × 7	42	3.53	52,733	34,606
1¼	6 × 6	36	3.41	53,248	33,520
1⅛	6 × 5	30	3.31	54,648	34,695
1	5 × 5	25	3.14	53,915	36,287

* These two tables of results are compiled from data given in the report of the U. S. Board on Testing Iron and Steel, vol. I, 1881.

CHAPTER IX.

STEEL.

METHODS OF MANUFACTURE.

96. The Crucible Process is the oldest and simplest of those used at the present time, and is still used for the finer grades of tool-steel. A pure grade of wrought iron is first rolled into flat bars and cut into convenient lengths. These are then heated for from three to six days in "cementing furnaces," where they are tightly enclosed in boxes separated by layers of fine charcoal. This recarburizes the wrought iron at the rate of about $\frac{1}{8}$ inch in depth every twenty-four hours, and makes *cement* or *blister* steel.* This was the steel of commerce until 1740, when it was first remelted in crucibles (by Daniel Huntsman, in England), thus making what is still known as *crucible* steel. These crucibles are now heated in a Siemens regenerative gas-furnace, simliar to that described in Art. 98. Cheaper grades of crucible steel are made by remelting in crucibles Bessemer scrap. The cheaper *Bessemer* and *open-hearth* processes have now limited the use of the crucible process to the manufacture of high-grade tool and spring steel only. In 1896 the total annual capacity of crucible-steel furnaces in the United States was about 100,000 gross tons.

97. The Bessemer Process.—This consists of a decarburization of crude pig iron by means of finely divided air-currents blown through the iron when in a melted state. The oxygen in the air burns out the silicon and carbon from the melted cast iron, and this combustion so raises the temperature of the melted mass that it remains a mobile fluid even after these foreign ingredients have been almost wholly removed. This requires a very high temperature indeed, and one which could not be obtained in the ordinary puddling-furnace. The purified iron is then "recarburized" by adding melted spiegeleisen which contains from 10 to 20 per cent of manganese, and also some carbon and silicon. This manganese unites with the large amount of oxide of iron present, which was formed by the blast, and which would cause the product to be red-short and to crumble in working, and

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* Also called shear, double shear, or German steel.

at the same time the proportion of carbon is brought up to any desired amount. The whole mass is then poured off into ladles, and thence into cast-iron moulds. These masses of cast steel are now called *ingots*. This process was invented by Sir Henry Bessemer of England, and perfected by G. F. Goransson of Sweden,* in 1858.

In this process the crude melted iron is tapped directly from the cupola furnace, and in Sweden directly from the blast-furnace into the converter, which is a large steel vessel, mounted on trunnions, lined with refractory materials, with a removable bottom provided with many small openings or tuyeres. This vessel is turned down into a horizontal position to receive its charge. The blast is then started and the vessel raised to a vertical position, the air-pressure being sufficient to keep the melted iron from entering the air openings in the base. In Sweden, where a very pure iron is used, the blast is stopped when the appearance of the shower of sparks issuing from the mouth of the converter indicates the

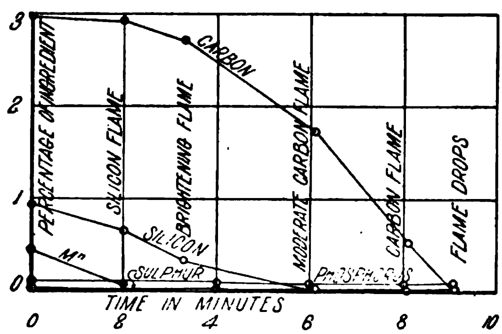

FIG. 62.—Chemical Reductions of an Open-hearth Converter. (Howe, *Jour. Ir. & St. Inst.*, vol. II. p. 102.)

desired percentage of carbon, when the metal is at once poured off into the moulds. As this criterion is a very uncertain one, it is customary in this country to continue the blast till practically all the carbon has been consumed, this stage being clearly indicated by the changed appearance of the flames. The addition now of manganese and carbon, in any desired proportions, is readily made. It is important to remember that by this process *no sulphur or phosphorus is removed*, and hence only pig irons comparatively free from these elements can be used for the Bessemer process, such iron being known as Bessemer pig. The iron must contain from 1½ to 2½ per cent of silicon in order that by its combustion it may sufficiently heat the charge to keep it fluid when the carbon is consumed. If there is as much as 2½ per cent of silicon in the pig-iron, from 10 to 15 per cent of

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* See paper by Prof. Rich. Ackerman of Stockholm, in *Trans. Am. Soc. Min. Engrs.*, vol. XXII. p. 266.

cold steel scrap can also be worked into the charge without chilling it. The rate of burning the silicon, carbon, and manganese is shown in Fig. 62. The combustion of the silicon brings to the mass about nine times as much heat as the combustion of the same amount of carbon. One reason for this is that the products of the combustion of silicon form a slag which remains

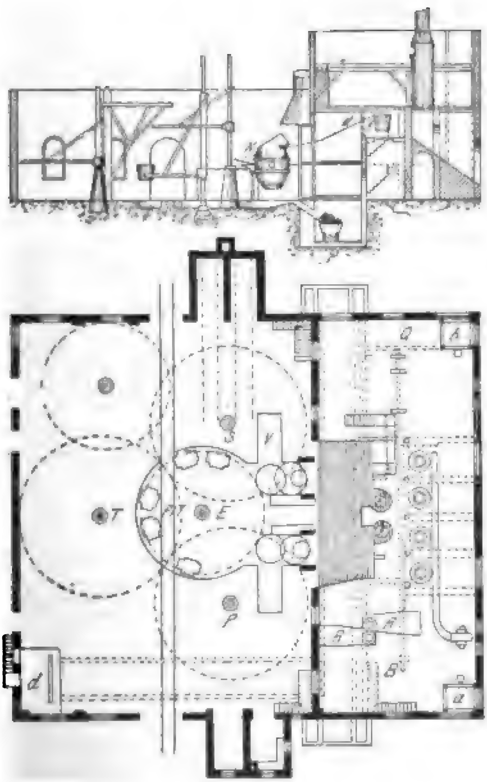

FIG. 63.—Plan and Sectional View of a Bessemer-steel Plant.

in the converter, while the product of the combustion of carbon is a gas which passes off and carries much heat with it.

In Figs. 63 to 67 are shown the characteristic features of a standard American Bessemer-steel plant. On the right of Fig 63, in plan, are shown four cupola-furnaces, with a blower, for melting the pig iron. The sectional view shows these to be placed at a high elevation, so that the melted iron, received in the ladles *K*, which stand on platform-scales for weighing the charge, can be poured into the spouts *MN*, and run directly into the mouth of the converter, which is then turned into the position shown in Fig. 65. The blast is now started through the base of the converter, and it is raised to a vertical position, Fig. 66, and the blast kept on

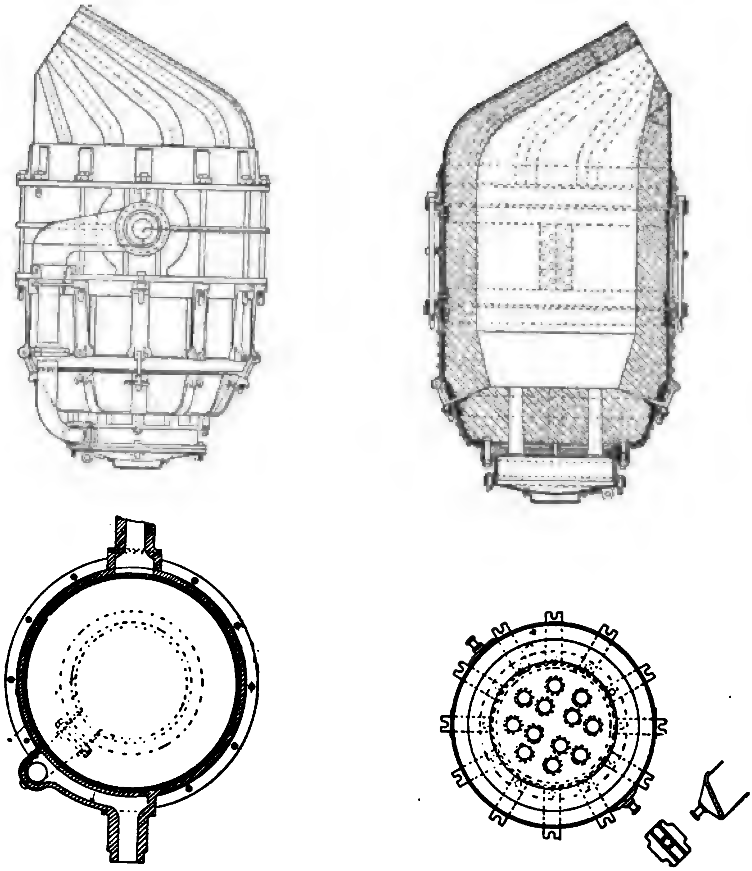

FIG. 64.—Views of the American Form of Bessemer Converter, showing the Movable Bottom.

FIG. 65.—Receiving the Charge.

till first the silicon and then the carbon has been burned out. The converter is then again revolved to a horizontal position, and the blast stopped. The proper amount of melted spiegeleisen which is kept melted in the two reverberatory furnaces *RR* is then run into the converter, whereupon it is at once poured into the ladle, which is operated by a crane which swings it in

FIG. 66.—The Bessemer Converter in Action.

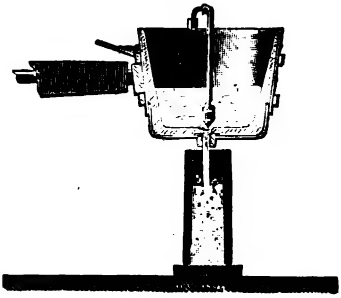

FIG. 67.—The Valvular Ladle.

the path of a circle over the several ingot-moulds, the metal falling through a valve at the bottom, as shown in Fig. 67. The motions of the converter and also of the crane, as well as the blast, are all controlled from one platform by levers operating hydraulic machinery.

The Swedish practice of taking the iron directly from the blast-furnace is growing in this country, but it is here first run into a large vessel containing from 100 to 150 tons of melted iron, called a mixer, in order to obtain a more uniform product. This mixer also serves as a reservoir for equalizing the inequalities of supply from the blast-furnace and demand from the converter. From this mixer it is drawn into ladles on cars, and run to an elevated platform and poured into the converter. This is called the direct process. It may be employed when the blast-furnaces are removed from the Bessemer plant as far as one or two miles.

Recently a means of removing phosphorus has been found in the addition of calcined lime to the charge in the converter.* The phosphorus unites with the lime and so passes into the slag. In this case the lining of the converter must also be "basic" to keep the slag from uniting with it and so rapidly consuming it, so the lining is then made of a calcined magnesian limestone (dolomite) and tar, made into brick, or rammed into place. This is called the *basic Bessemer* process, but its use had been abandoned in America because of some unfortunate failures when first introduced.

* By S. G. Thomas and P. C. Gilchrist, England, 1878.

The method has now (1896) been revived at Troy, N. Y., with marked success.

The Bessemer is the cheapest known process of making steel. This process alone has revolutionized many lines of industry, and has led to the replacing of wrought iron by steel in all the more important uses of these materials. The Bessemer process is now used exclusively for making steel rails for steam and electric roads, and for all the cheaper grades of steel plates and structural forms. For the better grades of structural material it is being replaced by—

98. The Open-hearth Process.—In this process pig iron, cast iron, and wrought-iron and steel scrap are converted into steel under the direct action of an oxidizing flame in a regenerative gas-furnace. It was patented in 1845 by Heath, but was not found to be successful until Siemens had developed his regenerative gas-furnace about 1862. Since about 1870

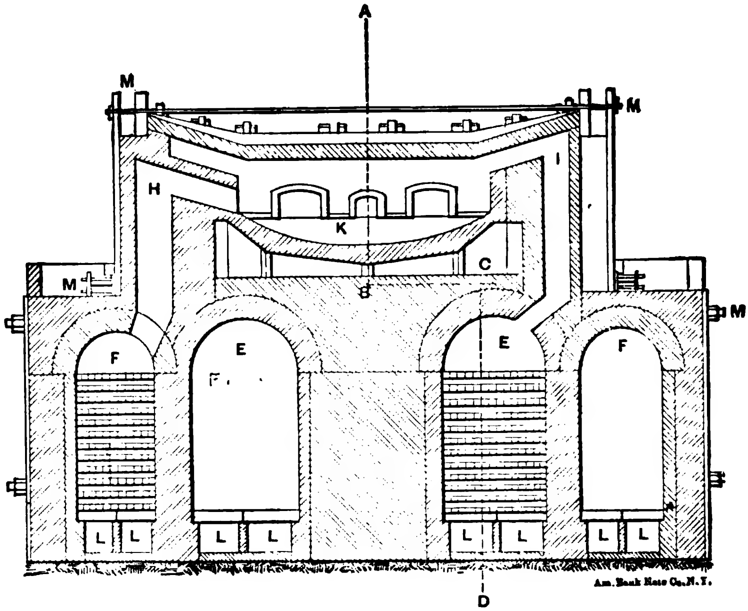

FIG. 68.—Transverse Section of a Typical Open-hearth Regenerative Gas-furnace.

these furnaces have multiplied rapidly, and in 1896 the total capacity of these furnaces in the United States was 2,400,000 gross tons, as against a total capacity of 9,400,000 gross tons by the Bessemer process.

The more common type of furnace used for this purpose is shown in Figs. 68 and 69. The fuel used is what is known as *producer-gas*. This is a mixture of carbonic oxide and hydrocarbons, diluted with about 60 per cent of nitrogen. It is formed in *gas producers* in which coal is burned in air-tight ovens with an insufficient supply of air, this supply being fed in

under pressure and in known volumes. This producer-gas is brought to the hearth area of the open-hearth furnace through a passageway entirely filled with red-hot fire-brick stacked to form an open checker-work, as shown at *E* and *F* in Figs. 68 and 69.

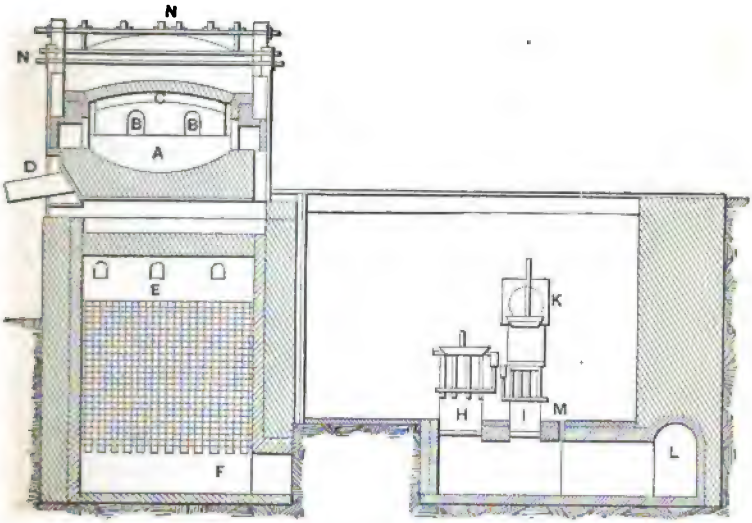

FIG. 69.—Longitudinal Section of a Typical Open-hearth Regenerative Gas-furnace.

As this hot gas enters the furnace area it is mixed with streams of hot air which has also been drawn in over red-hot brick surfaces, and the commingling of these red-hot gases, in proper proportions to produce complete combustion, develops the most intense heat possible to obtain by the combustion of gases. In the reverberatory furnace, where the flame of an ordinary coal fire is employed, the maximum temperature attainable is about 3,500 degrees F., but in the Siemens regenerative gas-furnace a temperature of 4500 degrees F. may be maintained. The *regenerative* principle consists in the utilization of the heat of the escaping gases in reheating the fire-brick placed in the air and gas passageways. To do this it is of course necessary to alternate the incoming and the escaping gases in two sets of passages, this being done by simply moving certain valves every twenty or thirty minutes.

In Fig. 68 *K* is the furnace hearth; *EE* are air-chambers and *FF* gas-chambers, the checker brickwork being shown in one only of each, but it really fills all four of these passageways. The red-hot gas enters the furnace through the lower ports *H*, Fig. 68, and *BB*, Fig. 69; while the air enters just above these through an annular space *I*, Fig. 68, and *C*, Fig. 69. The furnace itself, therefore, is like a great argand gas-burner in its method of receiving and burning the gas. The depressed roof of the furnace throws the heat strongly upon the materials placed on the hearth,

while the gases themselves are forced to play upon the melting metal. The flame has an excess of oxygen so that it is an oxidizing flame, and would rapidly waste unmelted wrought-iron or steel scrap placed in it. It is customary, therefore, to place first on the hearth pig or cast iron on which the oxidizing flame acts by consuming first the silicon and then the carbon, at
M, 73
the same time oxidizing some iron which by melting forms a slag which floats on the bath of melted metal. After such a bath has been prepared the wrought-iron and steel scrap can be thrown in, since these will now be covered by the bath and so protected from the oxidizing flame. The facility with which such scrap can be remelted and made over into new ingots by this method is one of its chief elements of value. If one could always choose his ingredients at pleasure he could so proportion them that little or no decarburization would be necessary, a simple melting together giving the requisite proportions.

The final removal of any excess of carbon, after the products have melted, is effected by means of the melted oxide of iron, which floats on the surface at first, but which afterwards becomes thoroughly mixed with the mass by the boiling action of the escaping gases when the temperature becomes high. Some of the oxygen of this iron oxide combines with the carbon of the melted iron and comes to the surface as carbonic oxide, where it is burned to dioxide and passes out with the other consumed gases. This also restores a corresponding portion of the iron of the oxide slag to the metal bath and so adds to the product.

When a large amount of pig or cast iron is to be reduced, it is common to charge a suitable amount of oxide of iron ore to supply the requisite amount of oxygen to decarbonize the cast iron, and so to hasten the process and also to avoid the necessity of creating so much artificial oxide of iron by the oxidizing flame. The remaining portion of the oxide slag not destroyed by giving up its oxygen to the carbon in the bath is neutralized and chemically destroyed by adding a charge of spiegeleisen containing 20 or 30 per cent of manganese, or an artificial ferromanganese containing some 80 per cent of manganese, just before pouring. The manganese unites with the oxygen of the slag, and restores the iron to the bath the same as is done in the Bessemer process. There, however, it was usually desired to add carbon also, and hence this process has come to be known as *recarburization*, or the adding of a *recarburizer*. In the open-hearth process the carbon is not all burned out, as it is in the Bessemer process, but by taking samples out with a small dipper or ladle, and casting these, cooling them in water, and breaking them, the operator can tell when he has the carbon ingredient brought to the desired amount. He seldom wants to add carbon, therefore, in the open-hearth process, but must add the manganese to destroy the oxide slag which, if poured off with the iron, would make it red-short, or unmalleable in the rolls. The manganese charge should more properly be called a "deoxidizer," but, by analogy from the Bessemer process, the same term of "recarburizer" is applied here to this manganese charge.

The pervasive action of this manganese charge in the open-hearth furnace is very remarkable. The manganese has so strong an affinity for the oxygen in the iron oxide that it seems quickly to seek it out throughout all parts of the bath, and even when added to the metal in the ladle, after teeming, it seems to be equally effective. In the case of the Bessemer process, however, it is necessary not only to destroy the iron oxide, but to add a carbon ingredient to the metal. To secure a uniform distribution of this carbon element through the mass seems to require a thorough artificial mixing. The only action of this kind which is secured in the Bessemer process is had in the pouring off into the ladle and the drawing from the pouring-nozzle in its bottom into the ingot-moulds. This does not insure a uniform distribution of the carbon, and hence it is not very uncommon to find great differences in the mechanical qualities of different portions of the same sheet of Bessemer steel. In the open-hearth process little or no carbon is added in the "recarburizer," so that the mixture retains the homogeneity it necessarily secures from the violent boiling action of the bath. This greater homogeneity and reliability, when judged by sample tests, has led to a general preference for open-hearth steel by engineers in all kinds of structural designing.

Here, as in the Bessemer process, there is no elimination of the phosphorus and of the sulphur. This has greatly limited the range of materials which could be fed to this furnace, and it has led here, as in the case of the Bessemer process, to the use of a charge of calcined lime to unite with the excess of phosphorus* and hold it in the slag, which is then drawn off. But, as with the Bessemer furnace, this lime would unite with the sand lining of the furnace to form a flux which would quickly destroy this lining altogether. To prevent this, those furnaces in which lime is to be added to the charge are themselves lined with calcined dolomite limestone, and these are called basic-lined furnaces, and this process has thus come to be known as the *basic open-hearth process*. It must be understood, however, that *neither here nor in the Bessemer process does the lining play any part in the process itself.* The process, in each case, depends on the materials charged and not on the furnace lining. The lining is simply made such as will not be attacked by the slag formed, and is always intended to be neutral, or inert. In 1896 about one half of the open-hearth steel made in the United States was by the basic process.

To distinguish the ordinary open-hearth process, in which a sand or silica lining is used and no lime fed in the charge, that is to say, in which no attempt is made to remove any of the phosphorus in the ingredients used, from this "basic" process, the former is now called the *acid open-hearth process*. It was formerly known as the Siemens-Martin process, from its use of the Siemens furnace and from the fact that the Messrs. Martin of France

* When steel is very low in carbon some phosphorus, as 0.03 or 0.04 per cent, seems desirable to add strength to the metal.

first employed the open-hearth in this way, but without the Siemens regenerative gas-furnace.

99. Comparison of the Basic and Acid Open-hearth Processes.—From what has been given in the previous article it is evident that poor steel and steel high in phosphorus *may* be made by either process, due to ignorance, carelessness, or inexperience. By the use of the basic process ingredients high in phosphorus may be employed, and thus the available materials are very much increased and hence cheaper grades can be employed. The process itself is somewhat more expensive than the acid process. When the acid process is used, unless there is a rigid inspection and control over the product, there is a danger that the cheaper (phosphorus) ingredients may be used, and so lead to a brittle product, whereas if the basic process be specified and employed the cheaper ingredients are anticipated, and the removal of the phosphorus provided for. The maker can be trusted to reduce the sulphur in order to make the product malleable when hot, so as to roll smoothly, as defects here are patent to any one. Engineers now generally specify the open-hearth process without prescribing the kind of lining, but naming a maximum proportion of phosphorus. This upper limit of phosphorus is now commonly taken at from 0.06 to 0.08 per cent, but Mr. H. H. Campbell,* who is the highest authority from the standpoint of the manufacturer, says this upper limit should now be made 0.04 per cent. If this phosphorus limit is not specified, or if specified but not determined by actual tests, then it would be safer to specify the basic open-hearth method.†

100. Comparison of Bessemer and Open-hearth Steel.—Comparing the products only (not the processes) of these two general methods of making steel on a large scale, we may say:

1. While for like chemical analyses like mechanical properties may be anticipated from these two methods, yet all the unexplained accidental failures of steel have occurred on Bessemer steel. Engineers have become suspicious of it. Open-hearth steel is therefore *more reliable* than Bessemer steel.

2. Test specimens cut from different parts of the same Bessemer steel plates have shown extraordinary differences in their mechanical properties. This has never been found in open-hearth plates. They are therefore *more homogeneous* than Bessemer plates.

3. Bessemer steel products found on the general market are apt to be extremely irregular in their composition, though rolled into like forms and sold to serve the same purposes. Open-hearth products purchased in the open market and designed to serve the same purposes are *more uniform* in quality.

4. The open-hearth steel may be tested before tapping off, and its composition adjusted at pleasure, and this is usually done. Bessemer steel

* Superintendent Pennsylvania Steel Co., Steelton, Pa.

† In January, 1896, there were in operation in the United States open-hearth-steel plants having an annual capacity of 2,430,000 gross tons, 700,000 of which capacity had been added in the preceding two years, more than one half of them using the "basic" process. Thirteen of these new furnaces are to be used in making steel castings.

TABLE XI.—TESTS SHOWING THE HOMOGENEITY OF OPEN-HEART METAL.

Heat 10,699. Acid Open-hearth. Test-bars, ¾″ rolled rounds.				Heat 10,910. Acid Open-hearth. Test-bars, 1¼″ rolled rounds.			
Elastic Limit, Lbs. per Sq. In.	Ultimate Strength, Lbs. per Sq. In.	Elongation in 8 In., Per Cent.	Reduction of Area, Per Cent.	Elastic Limit, Lbs. per Sq. In.	Ultimate Strength, Lbs. per Sq. In.	Elongation in 8 In., Per Cent.	Reduction of Area, Per Cent.
35,900	53,510	28.75	64.14	31,140	52,760	32.75	60.60
36,450	54,790	31.75	64.58	31,790	52,750	32.75	61.20
36,000	56,150	28.75	62.71	31,540	53,000	31.50	56.50
36,225	55,690	31.25	64.48	31,250	52,000	32.25	63.30
36,090	55,830	31.00	64.71	31,250	52,320	34 00	64.10
36,315	55,830	32.00	65.18	31,080	52,320	32.50	57.10
36.740	56,370	31.50	64.84	31,160	52,830	32.75	61.80
36.350	55,090	29.50	62.87	31,250	53,160	32.75	58.10
36,450	57,510	31.25	64.25	31,040	52,160	32.75	61.80
36,125	56,900	30.75	64.26	32,050	53,840	32.50	59.60
37,580	56,600	33.50	64.16	31,660	53,580	32.50	61.10
36.900	57.510	30.50	65.16	31,700	52,480	32.25	53.10
37,220	57,420	31.25	63.28	32,550	52,580	34.00	63.10
37,130	57,280	31.75	64.16	32,570	52,960	32.75	65.40
36,000	57,050	31.25	65.75	33,330	53,050	33.00	60.40
35.860	57,190	31.25	64.23	33,580	53,860	33.00	60.40
36 615	57,440	31.25	64.74	......	.....	....	....
36,450	57,670	31.75	66.46	......	......	....	....
37,165	57.580	32.75	63.68	......	......	....	....
36,640	57,350	31.25	63.18	......	......	....	....
Av. 36,510	56,538	31.15	64.34	Av. 31,809	52,853	32.75	60.48

Heat 11,018. Acid Open-hearth. Test-bars, ¾″ rolled rounds.				Heat 1,820. Basic Open-hearth. Test-bars, $\frac{7}{8}$″ rolled rounds.			
Elastic Limit, Lbs. per Sq. In.	Ultimate Strength, Lbs. per Sq. In.	Elongation in 8 In., Per Cent.	Reduction of Area, Per Cent.	Elastic Limit, Lbs. per Sq. In.	Ultimate Strength, Lbs. per Sq. In	Elongation in 8 In., Per Cent.	Reduction of Area, Per Cent.
36,700	58,400	28.00	66.20	33,065	48.340	34.50	71.87
37,150	57,840	30.75	65.60	31,530	47,380	35.00	72.05
37.280	56,880	29.50	58.60	33,650	48,450	35.00	72.05
36,060	56,940	28.50	53.30	31,600	48,230	37.00	74.14
36.420	56,700	30.75	63.10	33,340	49,175	36.25	70.09
36,060	57,180	30.25	65.50	32,760	48,560	33.75	79.25
35,780	56,800	31.00	65.40	33,260	47,730	35.00	74.49
36,700	57,440	30.00	63.80	32,130	48,785	34.00	71.80
35.780	56,800	31.00	65.40	32,935	48,640	34.25	71.92
36,700	57,440	30.00	63.80	33,270	49,440	34.00	71.48
35,700	56,900	32.50	67.10	32,900	47,835	34.00	72.72
37,020	57,180	31.25	59.70	31,920	48,050	33.75	71.42
37,400	57,320	30.50	68.10	32,185	48,360	36.25	74.28
37,260	56,780	30.25	67.20	33,880	48,400	33.75	73.64
37,480	57,420	31.00	66.30	......	.. ...	....	....
Av. 36,634	57,201	30.25	63.94	Av. 32,745	48,384	34.75	72.49

usually goes as blown, without correction. The open-hearth product is therefore *under better control.*

5. The remarkable homogeneity of open-hearth steel is indicated by the preceding series of tests (Table XI) on specimens cut from different portions of plates rolled from four different heats.*

101. Molecular Structure of Wrought Iron and Steel.—One of the most important facts for the engineer to fix in his mind is this: *All grades of iron and steel, originally formed in a molten state, will always thereafter, when cooled after either a melting, forging, or rolling, take a crystalline form.* That is to say, all cast, or ingot, metal is always crystalline whether cast, hammered, or rolled to its final forms.† This includes all the grades of "steel" as given in the second classification, p. 89, as well as the cast iron and cast steel.

It may also be said that *wrought iron also shows a crystalline structure whenever a portion of the iron in the puddle-ball was in a liquid condition when removed from the furnace.* This liquid iron may result either from the entangling of unreduced but melted cast iron in the glutinous mass, or from too high a temperature of the furnace, resulting in the melting down (burning) of the reduced metal, or wrought iron proper, which when "brought to nature" and at the proper temperature should be a spongy, pasty mass, sufficiently firm to be handled with the puddling-bars. This is only a special case of ingot metal, since so much of the iron in the puddle-ball as comes out in a liquid form is, within itself, free from the slag which covers the more pasty iron mass, within and without, as a slime might adhere to the entire internal and external surface of a sponge which has been lifted from it and squeezed. Any given portion of this puddle-ball, when finally rolled out into plates and bars, will become a small filament or fibre of the cross-section, but greatly extended in the direction of the rolling. Thus, if a small pocket of liquid iron was entangled in the ball, this would become a small crystalline thread throughout the bar. A larger mass of melted metal would make a longer crystalline portion of the cross-section.

When wrought iron is properly made, that is, when it is entirely reduced or "brought to nature," and when the furnace is not so hot as to melt the pasty mass, wrought iron will be found to be practically free from crystalline formations, and to be wholly fibrous. The fibrous structure is due to the continuous mixture of the slag with the iron, which, after repeated piling and rolling, leaves the slag so distributed through the mass in thin filaments as to prevent any visible crystalline arrangement of the molecules, although each such filament is really a series of distorted (usually elongated) crystalline forms.

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* From H. H. Campbell's paper on "The Open-hearth Process" before the World's Engineering Congress. *Trans. Am. Inst. Min. Engrs.*, vol. XXII. p. 352.

† That is to say, when shaped while hot. When shaped cold, as by cold rolling or wire-drawing, the crystalline form may be partly or even wholly destroyed.

102. Fracture Showing Structure.—In order to obtain a normal fracture of any malleable metal, or one which shows the true character of the molecular arrangement, uninfluenced by the distorting effects of the forces used to produce the fracture, it is common to nick the specimen with a chisel and bend it.* This, however, subjects one side of the uncut section to a compression, and the other to a tension; and even if a fracture is effected, the entire surface has not been treated alike. It is better, therefore, to turn a sharp groove into the side of the specimen all around (or nick it with a chisel), and then to pull the specimen in two in a testing-machine. This will always reveal the true structure of the metal, without the distorting effects accompanying the cold drawing out (elongation) of the specimen which is purposely sought in the ordinary tensile test. To insure against any elongation whatever, the tool used must be perfectly sharp at the point, so that the bottom of the groove is a true angle, and not a curve with a finite radius. When good structural steel is tested in tension it elongates at the ruptured section fully 100 per cent; that is, it stretches here to more than twice its original length, and this cold drawing out of the metal wholly destroys its original molecular arrangement, so that the fracture always look "fibrous" or "silky." This universal appearance of soft and medium steel when pulled in two leads many persons to suppose that this material has not normally a crystalline arrangement. When, therefore, they find this same material broken in use, as on a screw-thread, or at a shoulder, or at a sudden reduction of section, and they discover it having a wholly crystalline structure, they conclude that this is abnormal, and that the material has "crystallized in service." The simple test described above, of pulling a grooved specimen, will prove that all grades of steel, even to the softest ingot iron, has normally a crystalline structure. If the nicking test be cited to prove this fact, it is often claimed that the nicking produced such a jarring of the metal as to cause it to instantly rearrange its molecules, while cold and rigid, into the crystalline form! Surely the grooving in a lathe is not open to even this shadowy ground of suspicion.

If wrought iron be grooved and pulled as here described, it will be found to be apparently wholly fibrous (if of a superior quality, the crystallized filaments being so small), or containing occasional small crystalline patches, (if of an ordinary quality), or sometimes nearly wholly and coarsely crystalline (if of a very inferior quality). It is therefore said to be normally of a fibrous or non-crystalline structure. Now when wrought iron breaks in service and reveals a coarsely crystalline structure, it simply indicates, in the opinion of the author, the original poor quality of the material, and does not prove that the material had crystallized in service as is generally sup-

* Metcalf affirms that a skilful workman can grade steel, by the fracture, for customers, so closely that "year after year not one piece will vary in carbon more than 0.05 per cent above or below the mean for that temper." *Steel*, p. 6. This can only be true of material from the same establishment produced under like conditions.

posed. This subject has been discussed at considerable length in Article 93.

103. Structure of Steel as Affected by Heat Treatment.—While steel or ingot iron is entirely free from slag and similar foreign ingredients, it must not be regarded as a simple or single mineral, or substance, but rather as a substance, like granite, made up of a number* of separate minerals or "metarals" (a term suggested by Howe), each crystallizing out by itself, or being left as a matrix after the more controlling minerals have crystallized out. The particular final arrangement depends on which of these various proximate combinations control in the crystallizing stage of cooling, and also on the heat treatment it receives. Concerning the effect of the heat treatment on the appearance of the fracture, the following statements are based on the discussion of this subject by Howe (§§ 240–250). (See also Appendix A.)

1st. There is a critical temperature, at a "low yellow" heat (lower for high-carbon and higher for low-carbon steel), above which the material forms rapidly into coarse crystals.

2d. If cooled either slowly or rapidly from above this temperature, it is coarsely crystalline, the coarseness of the crystals depending on the time allowance for their formation when at this higher temperature.

3d. If worked (forged or rolled) in cooling from this higher temperature, the crystallization is that characterizing its temperature when leaving the hammer or rolls.

4th. If *raised* from a temperature below a low red, just to this critical temperature, whatever its previous condition, or if worked down to this critical temperature in cooling from a higher, and cooled rapidly as by quenching in water or oil, it is so finely crystalline as to appear amorphous, or porcelanic, to the naked eye. If cooled slowly from this critical temperature, it is finely crystalline to the naked eye.

5th. Since there is no tendency to crystallize below a low red heat, it is sufficient to cool rapidly from the critical temperature (low yellow) down to a low red, or to continue to work the metal to this temperature, after which the cooling may be slow, thus preserving the porcelanic fracture and obtaining a greater toughness.

6th. As an illustration of the effects of these different treatments, we have †

For slow cooling after forging, size of grain.............	0.1414 in. diam.
Reheated to the low yellow and cooled slowly, size of grain	0.0048 "
Reheated to low yellow, quenched in water to low red, and then slowly cooled, size of grain................	0.0004 "

* Seven such having already been distinguished; see Howe's *Metallurgy of Steel*, § 237. Osmond has recently added at least two new ones, and furthermore diamond has now been isolated from certain steels. *Stahl u. Eisen*, vol. xvi, No. 15, 1896.

† Quoted by Howe, § 250, from Chernoff.

The size of this last is entirely too small to be discoverable by the naked eye, and hence it would appear amorphous or porcelanic.

7th. The bright surfaces observed on a steel fracture may be either cleavage planes across individual crystals, or their exterior sides, depending on which surface offers the least resistance to rupture. In either case the size of these individual surfaces is a true index of the coarseness or fineness of the crystalline structure.

8th. The appearance of a steel fracture is thus a good indication of the condition of the metal when it left the rolls, or of its subsequent treatment. The student should himself verify these statements at the forge.

This subject will be discussed again when treating of hardening, tempering, and annealing. See Arts. 130 to 134.

104. The Mechanical Qualities of Steel.—When the term *steel* is made to include all grades of ingot metal as well as converted wrought iron, its qualities are so various as to necessitate a series of trade names, such as flange-steel and shell-steel, for boiler-plates; tank-steel, for plates of uncertain quality and of cheap manufacture, often used where a better grade is needed; structural steel, both mild and medium, used for all kinds of structural shapes, as angles, I beams, channels, T's, etc.; rail-steel, used for railway rails, both steam and electric; machinery-steel, especially adapted to forging and welding; tool-steel, spring-steel, saw-steel, etc.

The special qualities required of these various grades of steel are approximately as follows:

Fire-box Steel, Flange-steel, and Rivet-steel—used for locomotive fireboxes, boiler-heads, rivets, and other purposes where it is subject to great deformations in service, or where it must be shaped, or dished, in manufacture in such a way as is only practicable with a very soft, or pliable, semi-plastic material. This grade of steel, therefore, must be ductile rather than strong, and extremely tough and capable of resisting great abuse, either cold or hot, without losing its strength or toughness. This steel has a tensile strength of from 50,000 to 60,000 lbs. per square inch; an elastic limit* of from 30,000 to 40,000 lbs. per square inch; an elongation of from 25 to 35 per cent in eight inches; a reduction of area of from 50 to 65 per cent at the fractured section. It will also bend cold through 180, and close down perfectly flat, as in Fig. 71, either under the hammer or in a press, up to a thickness of ⅝ inch to 1 inch without showing any sign of fracture. One could literally tie it in knots while cold, as in Fig. 70 without sign of rupture. Made now mostly by the open-hearth process.

Shell-steel is used for boiler-shells, and for structural purposes, where a greater tensile strength may be obtained at the expense of some ductility. This steel has a tensile strength of from 55,000 to 65,000 lbs. per square inch; an elastic limit of from 33,000 to 44,000 lbs. per square inch; an elongation of from 25 to 30 per cent in eight inches; a reduction of area of

* Here the commercial elastic limit is meant, or the break-down point.

from 50 to 60 per cent at the fractured section. Made by the open-hearth and the Bessemer processes.

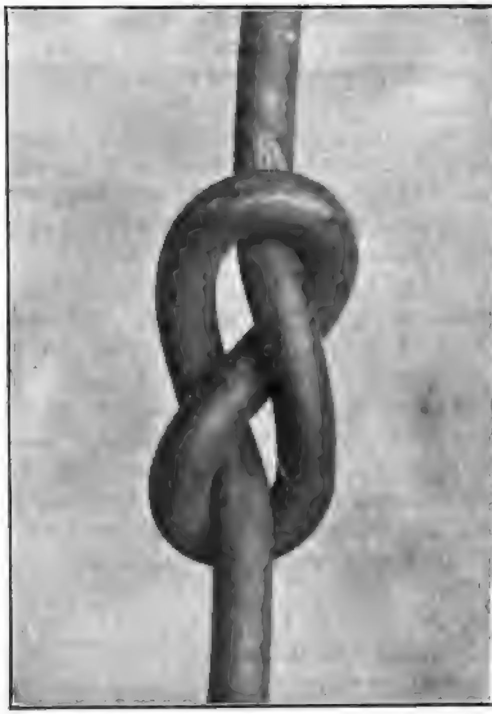

FIG. 70.—Knot of Rivet-steel, $\frac{7}{8}$ in. in diameter, pulled to Incipient Fracture by the Author.

FIG. 71.—Flange-steel Plates, $\frac{3}{8}$ in. thick.

Tank-steel has no particular limits of quality. It is a term which means the cheapest grade of steel plate on the market; is sold with no guarantee; its qualities usually unknown, or at least unrevealed; is likely to be extremely

various in quality, even in different parts of the same plate; and should be used only for indifferent purposes. Made by the Bessemer process. *

Structural Steel is used for bridges, roofs, steel skeletons of buildings, etc., and should be of a superior quality. Several grades are recognized, although these names are used loosely, and have no precise meaning. Thus *soft* and *mild* structural steel may be regarded as the same in quality as the flange and shell steel respectively described above. *Medium* structural steel might be considered as having a tensile strength of from 60,000 to 70,000 lbs. per square inch; an elastic limit of from 35,000 to 45,000 lbs. per square inch; an elongation of from 20 to 25 per cent in eight inches; a reduction of area of from 50 to 60 per cent at the fractured section. *Hard* structural steel having a tensile strength of from 65,000 to 75,000 lbs. per square inch is now used but little, as it suffers too much from shearing, punching, and assembling to make it as reliable as is desired for such material. Made by both the open-hearth and the Bessemer processes.

Rail-steel must be very hard, with a high elastic limit to resist abrasion and wear, while it must also have great strength and resilience, or resistance to shock. It is a hard steel, having a tensile strength of from 70,000 to 80,000 lbs. per square inch; an elastic limit of from 40,000 to 50,000 lbs. per square inch; an elongation of from 15 to 20 per cent in eight inches; a reduction of area of from 40 to 50 per cent at the fractured section. Made by the Bessemer process.

Ordinary Tool-steel, *Spring-steel*, etc., are harder grades, capable of being hardened and tempered, in which the tensile strength and ductility are of less importance than its hardening qualities. The tensile strength here may be from 90,000 to 160,000 lbs. per square inch and the elongation very small, depending on the particular temper given to the specimen. Made by the Bessemer, open-hearth, and crucible processes.

The Finer Grades of Tool- and Spring-steel, especially such as are to be used for edge-tools, are still made of crucible-steel. Metcalf gives the following tempers for their respective uses:

"0.50 to 0.70 C for hot work and for battering-tools, and for tools of dull edge.

"0.70 to 0.80 C for battering-tools, cold-sets, and some forms of reamers and taps.

"0.80 to 0.90 C for cold-sets, hand-chisels, drills taps, reamers, and dies

"0.90 to 1.00 C for chisels, drills, dies axes, knives, and many similar purposes.

"1.00 to 1.10 C for axes, hatchets, knives, large lathe-tools, and many kinds of dies and drills if care be used in tempering them.

"1.10 to 1.50 for lathe-tools, graving-tools, scribers, scrapers, small drills, and many similar purposes.

"The best all-around tool-steel is found between 0.90 and 1.10 C. This can be adapted safely and successfully to more uses than any other temper."

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* See Plate III for the sad effects of using such material in an important structure.

This is at and just above the point of complete saturation of combined carbon.

105. Qualities of Steel as Affected by its Chemical Composition.—Mr. H. M. Howe, the highest authority on this subject, says (§ 1): "I conceive steel to consist (A) of a matrix of iron which is sometimes (as in ingot iron and annealed steel), comparatively, or even quite pure, and sometimes (as in hardened steel, manganese-steel, etc.), chemically combined with a portion, or even the whole of the other elements which are present, probably in indefinite ratios, its mechanical properties being greatly affected by them; and (B) of a number of independent entities which we may style 'minerals,'* chemical compounds of the elements present, including iron, which crystallize within the matrix, and by their mechanical properties, shape, size, and mode of distribution also profoundly affect the mechanical properties of the composite mass, though probably less profoundly than do changes of corresponding magnitude in the composition of the matrix."

And again (§ 237), "From the microscopic study of polished sections iron (and steel) appears to be constituted, like granite and similar compound crystalline rocks, of grains of several distinct crystalline minerals, of which seven common ones have already been recognized, through peculiarities of crystalline form and habit, color, lustre, hardness, and behavior towards solvents. Their nature, size, shape, and orientation, and through these the structure and physical properties of the metal as a whole, seem to depend chiefly—

1. On the ultimate chemical composition of the mass;
2. On the mechanical treatment which it has undergone;
3. On the conditions under which it has been heated and cooled, i.e., its "heat-treatment," which may induce the ultimate components of the mass to regroup themselves in new combinations, thus causing one set of minerals to give place to another."

When the iron or steel is in a state of fusion the ingredients are in mutual solution, and they do not separate until the fluid mass congeals or hardens, when one or another of these mineral ingredients crystallizes out first, and thus gives its own characteristics greater prominence than the other minerals which form the matrix, or which form in crystals later, and subject to the limitations as to form and size imposed by the previously formed crystals.

Just as the character of a granite rock, therefore, is to be judged from the character of its mineral constituents, as proximate chemical compounds, and very imperfectly from ever so exact a determination of its ultimate elements, so we must learn to rely with less assurance on the ultimate chemical analysis of iron and steel, and more on the proximate chemical compounds formed therefrom. Unfortunately these latter are very difficult of determination, or even of identification, and hence we know very little about them. It is for this reason that we are as yet unable to infer with any great

* But for which Mr. Howe suggests the term "metarals."

assurance the mechanical properties from the chemical analysis. Such conclusions as may be drawn from chemical composition are partially summarized in the following articles.

INFLUENCE OF CARBON ON IRON.

106. Combination of Carbon with Iron.—The effects of carbon on iron are more pronounced and useful than those of any other known chemical element. Iron absorbs carbon readily, becoming saturated with about 4.6 per cent of it, unless aided by manganese, when it may absorb as much as 7 per cent.

Cast Iron may be regarded as supersaturated with carbon, or as having some 4 per cent of this element in some form.

Wrought Iron is nearly free from carbon in any form, having perhaps not over 0.10 per cent.

Steel (ingot metal) may have anywhere from 0.05 to 1.50 per cent of carbon, the upper limit usually being about one per cent. For extreme hardness, steel may be made with as much as two or even three per cent carbon, while 0.9 per cent C gives maximum working qualities for tool- and spring-steels. (This is the point of perfect saturation of combined carbon.)

Three States of Carbon in Iron.—Carbon is found in iron in three radically different states:

1. Mechanically mixed, in the form of a *graphite*, this being thrown out, or excluded, when cast iron crystallizes from a melted state.

2. Chemically combined in unknown proportions, this forming a very hard and strong compound, and the carbon so combined being here called *hardening carbon.**

3. Chemically combined, as a carbide of iron (Fe_3C), up to the saturation-point of 0.9 C. It is intensely hard according to Sorby, though it does not appear to contribute to the hardness of steel to the same degree as the hardening carbon.†

We shall therefore speak of the uncombined carbon as *graphite* (often called graphitic carbon), and the chemically combined carbon as *hardening carbon* and *cement carbon.*

When the metal is fused all the carbon may be regarded as chemically combined in the form of hardening carbon. When there is a great deal of this, as in cast iron, a large proportion of it is thrown out as graphite in the early stages of cooling, if sufficient time be allowed for this action to complete

* Prof J. O. Arnold (Sheffield) gives the formula $Fe_{24}C$ for this component. *Trans. Inst. Civ. Eng.*, vol. CXXIII, 1896. Many authorities agree with Osmond in attributing the hardness of quenched steel to an allotropic form of iron. Both sides are well presented in the discussion of Arnold's paper, here cited. See Appendix A.

† Besides these diamond has recently been isolated, and Ledebur adds "temper-carbon."

itself. When there is not over 0.9 per cent of total carbon, none of it will appear as graphite in the cold product.

A change from hardening carbon to cement carbon occurs (time permitting) at a low yellow heat, and no further change occurs below a low red heat.

These changes and also the subsequent condition of the metal are indicated graphically, in a general way, in Figs. 72 and 72*a*. Thus in Fig. 72 the total carbon in cast iron being about 4 per cent, as soon as it begins to

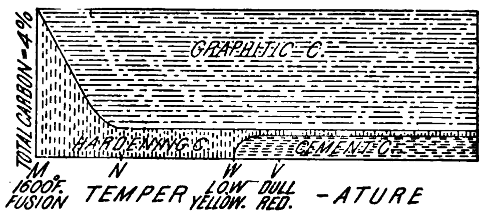

FIG. 72.—Change of Carbon in Cast Iron.

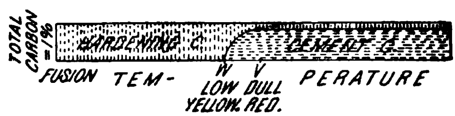

FIG. 72*a*.—Change of Carbon in Steel.

congeal or crystallize, it begins to expel carbon in the form of graphite, and this action is supposed to be completed when the metal has cooled to *N*, at which time the product has probably become wholly crystalline, having perhaps less than one per cent of carbon left in the combined form, as hardening carbon. It is new very granular in its nature, having little or no cohesion, and this intermediate granular form will always prevent the rolling of steel direct from the melted state. At the temperature *W* (low yellow) a peculiar change occurs in the combined carbon, a large part of it passing from the hardening to the cement form, if sufficient time be given at this temperature for this to occur. This change in the carbon state is accompanied by a remarkable development of sensible heat, causing the color to brighten up again, and this phenomenon is known as *recalescence*. This marks the truly plastic state at which it should be worked. As shown in Fig. 72*a*, there is no appreciable amount of graphitic carbon in steel, it all being in chemical combination, but changing from the hardening to the cement form, in a falling, and back again for a rising, temperature past the critical low yellow heat.

The presence of the large amount of graphitic carbon in cast iron causes it to fuse at a much lower temperature than steel, because of the recombining of this carbon, chemically, with the iron at this high heat. The fusing temperature of steel is higher as the proportion of carbon is less.

107. Physical Effects in Steel of the Change in the Combined Carbon at a Low Yellow Heat.—Hardening and Tempering.—This change in the combined carbon of steel from hardening to cement and back again is accompanied by a corresponding change in the crystalline arrangement, in the appearance of the fracture, and in all its mechanical properties. Thus if the region W—V, Fig. 71 be passed quickly, as when the specimen is quenched in water from a temperature above W, there is very little change in passing this critical temperature, and hence the carbon remains mostly in the hardening state. This gives a very hard and brittle product (when the percentage of carbon is high, or from 0.75 to 1.0 per cent), and in all cases raises the elastic limit and the ultimate strength,* but reduces the ductility. The crystalline arrangement, also, is now that which was formed on the first cooling, above W, it being very coarsely crystalline.

If the region W—V be passed slowly, more especially if the specimen be held at this temperature for a considerable period and then cooled slowly, the combined carbon changes mostly to the cement state, and a great softening of the material results. The only way to retain the carbon in the hardening state, when cold, being to cool it quickly from a temperature above W.†

When steel has been hardened by sudden cooling from above W, it can be tempered, or softened, by heating again, to some temperature below V and cooling slowly. The higher this tempering heat is, below a red heat, followed by slow cooling (as in the air), the softer will be the product when cold, as the more of the hardening carbon will be changed to the cement state. If the reheating be carried to V or above, and cooled slowly, the carbon will be (almost) wholly in the cement state, the temper then having been entirely drawn. The particular temper required, therefore, is obtained by first quenching from W or above, then reheating to the required temperature below V, and cooling slowly. This leaves the required portion of the carbon in the hardening state, and gives the product the desired compromise qualities of strength, hardness, and ductility, combined with toughness.

In the matter of the fracture, also, either a slow or a rapid cooling from a white heat, without forging or rolling, leaves a coarse crystalline fracture.

If worked down to a red heat it gives a fine crystalline fracture.

It is of the utmost importance that the heating for both hardening and for tempering should be uniform throughout the entire body of the specimen. Evidently a liquid bath of some kind furnishes the ideal condition for both heating and cooling. Thus a melted lead bath, kept stirred, may be used for heating, and a mercury, brine, water, or oil bath for the quenching, or sudden cooling. All hardening should be done by quenching from a *rising* temperature, to preserve fineness of grain. The reheating of the hardened

* Quenching in water from a high temperature may impair the ultimate strength of low carbon-steel. Quenching in oil seems always to increase the ultimate strength.

† If not uniformly heated when quenched, it is apt to break or crack from internal stress. The different densities resulting from quenching from different temperatures, may furnish a key to this action.

steel for the purpose of tempering it may be done by holding it over a fire, or in contact with a heated mass of iron, or in boiling water, or hot steam, or in some other way.

When clean iron or steel is heated in the open air, the oxide which forms on the surface takes in succession the following well defined colors, namely: light straw, straw, light brown, darker brown, pigeon-wing (a purplish brown), light blue, dark blue, and black. In tempering, the final "temper" depends on which of these graduated colors has been reached, and followed by slow cooling. Thus if only the first color indication, light straw, be reached, and then the bar slowly cooled, evidently very little softening of the hardened steel has resulted, and the product is left very hard, or it is said to have a "very high temper"; whereas if the highest temperature had been reached, at which the oxide had deepened to black, and then the bar cooled slowly, it would be found to be quite soft, or the hardness would have been entirely removed. The word "temper" then may have the following meanings, according as it is used by the steel-maker or by the steel-user:

Designation of "Temper."	Steel-maker's Meaning. Percentage of Carbon.	Steel-user's Meaning. Temper drawn at	
		Temperature.	Name of Color.
Very high...........	1.50 carbon	About 400° F.	"Light straw"
High...............	1.00 to 1.20 C	" 450° F.	"Straw"
Medium.............	.70 to .80 C	" 500° F.	"Brown" to "pigeon-wing"
Mild...............	.40 to .60 C	" 550° F.	"Light blue"
Low................	.20 to .30 C	" 600° F.	"Dark blue"
Soft, or dead soft.....	Under .20 C	" 650° F.	"Black"

EFFECTS OF CARBON IN ITS VARIOUS STATES ON THE MECHANICAL PROPERTIES OF IRON AND STEEL.

108. Not Fully Explained by Chemical Analyses.—As shown in Art. 105, the mechanical properties of iron and steel are not fully indicated by any ultimate chemical analysis of the material, but are dependent on the particular combinations the elements may have formed. In Art. 106 it was further shown that carbon is found in three distinct forms in iron and steel, and that the physical qualities depend largely on these particular forms of carbon. It is to be expected, therefore, that the mechanical qualities, or the qualities shown by the material when resisting the action of external forces, would also be found to be greatly dependent on these particular forms of carbon, combined and uncombined, or even on the total combined carbon, since this has been shown to exist in two very different states. The effort, therefore, of students of this subject to harmonize the results of mechanical tests with the corresponding ultimate chemical analyses of the materials was foredoomed to failure. And since the proximate chemical analysis is as yet impossible, we are wholly unable to predict mechanical properties from

chemical analysis alone. When this is supplemented, however, with a full knowledge of the heat treatment, as described in the preceding article, some approximate knowledge of the mechanical properties is obtained. (See also Appendix B.

109. The Hardening of Steel.—A coarsely crystallized steel may be reheated to a temperature between *V* and *W*, and cooled either slowly or rapidly, and the fracture becomes finely crystalline or even porcelanic. Just what does occur in the hardening of high carbon steel is, and has long been, a matter of contention among our most distinguished metallurgical chemists. Osmond and his school contend for an allotropic form of iron (called "β iron," to distinguish it from the annealed form, which he called "α iron"), not to be explained by a definite chemical compound, but containing carbon in solution, while Prof. Arnold makes a very strong plea for a chemical compound, $Fe_{24}C$, which he calls a "sub-carbide" (Fe_3C being the carbide), this being he thinks the real composition of steel in a melted state, having as much as 0.89 per cent C, which he calls the point of saturation.* When this compound is cooled suddenly, this unstable sub-carbide hardens into a solid without any change in its chemical composition; but when cooled slowly, it passes at 400° C. into the carbide form, with pure iron ($Fe_{24}C = Fe_3C + 21Fe$) and with the evolution of heat. See Plates in Appendix B.

Prof. Arnold gives, as a general summary of his views, the following:†

I. The constituents of steel may be: (*a*) Crystals of pure iron which remain bright on etching. (*b*) Crystals of slightly impure iron which become pale brown on etching, probably owing to the presence of a small quantity of an intermediate carbide of hypothetical formula $Fe_{10}C$. (*c*) Normal carbide of iron, Fe_3C, which exists in three distinct modifications, each one conferring upon the iron in which it is found particular mechanical properties. (1) Emulsified carbide present in an excessively fine state of division in tempered steels. (2) Diffused carbide of iron occurring in normal steels in the forms of small ill-defined striæ and granules. (3) Crystallized carbide of iron occurring as well-defined laminæ in annealed and in some normal steels. (*d*) Subcarbide of iron, a compound of great hardness existing in hardened and tempered steels and possessing the formula $Fe_{24}C$. This substance is decomposed by the most dilute acids, and at 400° C. it is decomposed into Fe_3C and free iron with evolution of heat. One of the most remarkable properties of this compound is its capacity for permanent magnetism. (*e*) Graphite or "temper-carbon."‡

The existence of $Fe_{24}C$ is proved by the fact that iron containing 0.89 per cent carbon presents several correlative critical points when examined by different methods of observation: (1) Well-marked saturation-points in the micro-structure of normal annealed and hardened steels. (2) A sharp maximum in a curve the coordinates of which are heat evolved or absorbed in recalescence and carbon percentage. (3) A point in the compression curve of hardened steels at which molecular flow absolutely ceases. (4) A sharp maximum in a curve the coordinates of which are carbon percentage and permanent magnetism in hardened steels.

II. The influence of annealing is—(1) To increase the size of crystals and to increase the intercrystalline cohesion when originally feeble or impaired. (2) To convert elongated masses of iron containing diffused Fe_3C into compact rounder

* With as much as 1 per cent manganese he claims the point of saturation with carbon is reached with 0.65 C. See Appendix B.

† *Trans. Inst. Civ. Engrs.*, vol. CXXIII, 1896, p. 160. See also Appendix B.

‡ Ledebur distinguishes between graphite and temper-carbon.

bodies, containing laminæ of crystallized Fe_3C, between which the iron becomes more or less dovetailed throughout the mass.

III. The approximate theoretical constituents of hardened and normal steels will be in accordance with the figures given in Table IX. (These percentages, however, can never be quite correct, because in practice hardened steels below the saturation-point (0.89% C) always contain a little Fe_3C, and normal steels below the saturation-point a small quantity of the intermediate carbide, $Fe_{10}C$(?)). It is obvious that in tempered steels an almost unlimited variety of constitutions and consequently of mechanical properties is possible.

TABLE IX.—APPROXIMATE THEORETICAL COMPOSITION OF HARDENED AND NORMAL IRON AND CARBON STEELS REQUIRED BY THE SUBCARBIDE THEORY HEREIN ENUNCIATED.

Carbon.	Hardened Steels.			Normal Steels.	
	Fe.	$Fe_{24}C$.	Fe_3C.	Fe.	Fe_3C.
Per cent.	Per cent.	Per cent.	Per cent.	Per cent	Per cent.
0.10	89	11	0	99	1
0.20	78	22	0	97	3
0.30	67	33	0	95	5
0.40	56	44	0	94	6
0.50	45	55	0	93	7
0.60	34	66	0	91	9
0.70	22	78	0	90	10
0.80	11	89	0	88	12
0.90	0	100	0	87	13
1.00	0	99	1	85	15
1.10	0	97	3	84	16
1.20	0	95	5	82	18
1.30	0	93	7	81	19
1.40	0	91	9	79	21
1.50	0	89	11	77	23

IV. The subcarbide theory falls into line with the observations of every-day experience. For instance, the fact has long been known that pure carbon steel, containing about 0.85 per cent of carbon, is the most suitable for steel which must carry a cutting edge and yet be tough enough to withstand a sudden shock. Such steel is therefore employed for cold sets.* It is also well known that a steel containing 1.3 per cent of carbon would be useless for such a purpose, as it would crack and "snip." The reason is clear; such material is full of lines of weakness along the junctions of the subcarbide granules with the surplus normal carbide membranes. On the other hand, it is known that a steel harder than one carrying 0.9 per cent of carbon is necessary for turning-tools. In such a case no shock has to be encountered, so that the surplus Fe_3C augments the hardness of the subcarbide with its own intense hardness, and moreover adds 10 per cent of a substance incapable of "letting down" with the heat of friction. It is also clear that a steel with carbon much below 0.9 per cent cannot carry a cutting edge, because of the presence of particles of soft free iron amongst the mass of the hard subcarbide. See App. B.

110. Effect on Tensile Strength.—While a diagram showing the relation of tensile strength to percentage of carbon in steel gives a cloud of results spread over a wide belt (see Howe's *Metallurgy of Steel*, p. 14), yet a simple formula which is the algebraic expression of a line which traverses this field well below its centre of gravity may be of some use. While many

* As required in saws.

such formulæ have been proposed, that of Salom* seems best to fit the total assemblage of results and is easily remembered. It is

$$T = 45{,}000 + 100{,}000C, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where T = tensile strength of rolled steel in pounds per square inch (up to C = 1.0 per cent);

C = percentage of carbon

The recorded tests show many results as much as 20,000 pounds per square inch above this locus, and some 10,000 pounds below it. It may be regarded, therefore, as traversing the lower edge of the middle third of the cloud of recorded observations. As the maximum strength of steel is reached with C = about 1.0 per cent, the above formula must not be used above $T = 145{,}000$ and $C = 1.0$. Higher values of tensile strength, as with drawn steel wire, are due to the physical treatment and not to the chemical composition. The elastic limit in both tension and compression may be taken as 60 per cent of the tensile strength.

As a result of a careful study of over four hundred tests accompanied by their corresponding chemical analyses, made in the regular course of business, Mr. William R. Webster† offers the following table, showing the variation of strength of soft steel with varying percentages of carbon and phosphorus, assuming the manganese and sulphur are each zero. When either or both of these latter are present, the values may be corrected by adding the values given in the two auxiliary tables for the corresponding percentages of these ingredients.

TABLE XII.—ESTIMATED ULTIMATE STRENGTH OF STEEL FOR VARYING PERCENTAGES OF CARBON AND PHOSPHORUS.‡

On the assumption that neither manganese nor sulphur is present, the tabular values to be increased for these ingredients by the amounts given in the two following auxiliary tables.

Carbon in Parts of 1 per cent.	.06	.08	.10	.12	.14	.16	.18	.20	.22	.24
Percentage of										
Phos. .000	39,550	41,150	42,750	44,350	45,950	47,550	49,150	50,750	52,350	53,950
" .01	40,350	41,950	43,750	45,550	47,350	49,050	50,650	52,250	53,850	55,450
" .02	41,150	42,750	44,750	46,750	48,750	50,550	52,150	53,750	55,350	56,950
" .03	41,950	43,550	45,750	47,950	50,150	52,050	53,650	55,250	56,850	58,450
" .04	42,750	44,350	46,750	49,150	51,550	53,550	55,150	56,750	58,350	59,950
" .05	43,550	45,150	47,750	50,350	52,950	55,050	56,650	58,250	59,850	61,450
" .06	44,350	45,950	48,750	51,550	54,350	56,550	58,150	59,750	61,350	62,950
" .07	45,150	46,750	49,750	52,750	55,750	58,050	59,650	61,250	62,850	64,450
" .08	45,950	47,550	50,750	53,950	57,150	59,550	61,150	62,750	64,350	65,950
" .09	46,750	48,350	51,750	55,150	58,550	61,050	62,650	64,250	65,850	67,450
" .10	47,550	49,150	52,750	56,350	59,950	62,550	64,150	65,750	67,350	68,950
.001 Phos. =	80 lbs.	80 lbs.	100 lbs.	120 lbs.	140 lbs.	150 lbs.	150 lbs.	150 lbs.	150 lbs.	150 lbs

*See *Trans. Am. Inst. Min. Engrs.*, XIV. p. 127, and also Howe's work as quoted above.

† *Trans. Am. Inst. Min. Engrs.*, vol. XXIII. p. 114.

‡Campbell gives, in his work on Structural Steel, p. 306:

Ultimate strength of acid steel $= 33{,}000 + 1485\,C + 1260\,P$,

" " " basic " $= 40{,}000 + 1085\,C + 1200\,P$.

where C = percentage of carbon and P = percentage of phosphorus.

TABLE XIII.—ADDITIONS FOR SULPHUR IN PARTS OF ONE PER CENT.

Sulphur	0	.01	.02	.03	.04	.05	.06	.07
Additions in pounds per square inch	.000	500	1000	1500	2000	2500	3000	3500

TABLE XIV.—ADDITIONS FOR MANGANESE IN PARTS OF ONE PER CENT.

Man.	Lbs.	Man.	Lbs.	Man.	Lbs.	Man.	Lbs.	Man.	Lbs.
.15	3,600	.27	6,300	.38	8,280	.49	9,780	.60	10,900
.16	3,840	.28	6,500	.39	8,440	.50	9,900	.61	11,000
.17	4,080	.29	6,700	.40	8,600	.51	10,000	.62	11,100
.18	4,320	.30	6,900	.41	8,740	.52	10,100	.63	11,200
.19	4,560	.31	7,080	.42	8,880	.53	10,200	.64	11,300
.20	4,800	.32	7,260	.43	9,020	.54	10,300	.65	11,400
.21	5,020	.33	7,440	.44	9,160	.55	10,400	.66	11,500
.22	5,240	.34	7,620	.45	9,300	.56	10,500	.67	11,600
.23	5,460	.35	7,800	.46	9,420	.57	10,600	.68	11,700
.24	5,680	.36	7,960	.47	9,540	.58	10,700	.69	11,800
.25	5,900	.37	8,120	.48	9,660	.59	10,800	.70	11,900
.26	6,100								

111. Effect on Ductility.—In general the ductility of steel diminishes as the percentage of carbon increases. The ductility is usually determined by dividing the total stretch of a specimen between marks eight inches apart, which includes the section of rupture, by the original length of eight inches. This total stretch is found after the specimen has been broken in tension, and is called the "percentage of elongation." From the plotted results of over one thousand determinations of elongation with known percentages of carbon,* the author of this work would express this general relation by the formula

$$E = \frac{3}{C^2 + 0.1}, \quad \ldots \ldots \quad (2)$$

where E = percentage of elongation in eight inches,

C = percentage of carbon (less than 1.00).

This would seem to give too low a percentage of elongation by about 5 per cent for carbon from 0.25 to 0.45 per cent. In any case the elongation may vary from the mean as given by this equation by at least one fourth of its value, showing that the ductility is dependent on other things besides the proportion of carbon.

There seems to be no difference between open-hearth and Bessemer steel in this respect, but crucible steel gives an elongation equal to that of the other varieties of a lower carburization. In other words, crucible steel is more ductile than the cheaper grades, for the same proportion of carbon.

* Howe's *Metallurgy of Steel*, p. 16.

112. Elongation and Tensile Strength.—Mr. Howe gives a table of the common greatest and least limits of elongation for various grades of steel, which have been plotted in Fig. 73. The shaded area between these limits may be regarded as the Elongation Field. In this field has been drawn two curves, which are the loci of two equations expressing elongation in terms of the ultimate strength. One of these (*a*) has been proposed by a committee of the Am. Soc. Civ. Engrs. (July, 1896), and the other (*b*) by the author.

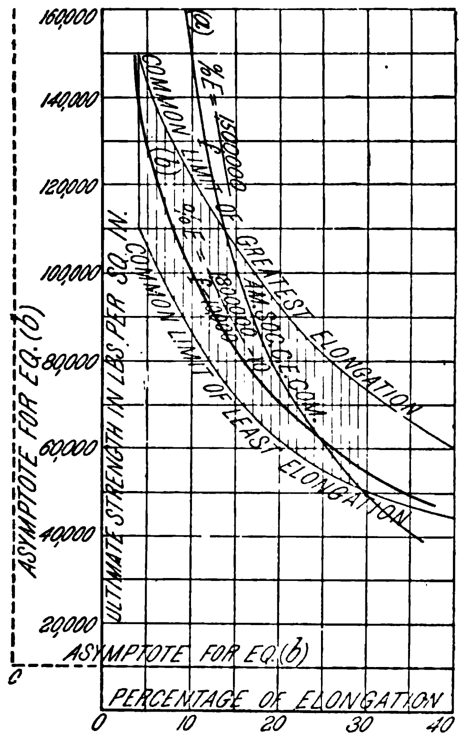

FIG. 73.—Showing the Elongation Field for Structural Steel and the Loci of Proposed Elongation Equations. Limits of Greatest and Least Elongations taken from Howe's *Metallurgy of Steel.*

The former is an equilateral hyperbola, making the product of the ultimate strength per square inch and the percentage of elongation a constant, and equal to 1,500,000, or

$$E = \frac{1,500,000}{f}. \quad \quad \quad (3)$$

The other is also a hyperbola referred to asymptotes parallel to the main axes, but removed from them, as shown in the figure, and whose equation is

$$E = \frac{1,800,000}{f - 10,000} - 10. \quad \quad \quad (4)$$

113. Modulus of Elasticity.—As stated in Art. 11, the modulus of elasticity is not appreciably affected by the percentage of carbon or by any other ingredient. This is also shown by Fig. 294. The author of this work believes that with such determinations of this modulus as have been made hitherto, it is rather to be presumed that discrepant values are due to inadequate or erroneous methods of determination rather than to actual wide departures of the modulus from its mean value.

114. The Compressive Strength.—The elastic limit is the real ultimate compressive resistance with the softer grades of steel, having a definite "yield-point" (see Fig. 294), while with hard steel there is no yield-point and no very definite elastic limit, and the ultimate strength in compression is clearly marked by a decided rupture on planes of maximum shearing stress. Few tests of steel have been made in compression, but it is shown in Chap. XXVI. that the compressive elastic limit is numerically equal to that in tension, or as 60 per cent of the ultimate tensile strength. In other words, the compressive resistance of steel is increased by increasing carbon the same as the tensile strength.

115. Hardness and Fusibility.—The hardness increases with increasing carbon apparently without limit.

The fusibility also increases with increasing carbon without limit. Thus cast iron and the hard grades of steel melt at a much lower temperature than wrought iron and the soft steels.

INFLUENCE OF SILICON ON IRON AND STEEL.

116. Combination of Iron and Silicon.—"Silicon alloys with iron in all ratios, at least up to 30 per cent, being readily reduced from silica (SiO_2) by carbon in the presence of iron. It rarely, if ever, exists in iron in the graphitoidal state. It diminishes the power of iron to combine with carbon, not only when molten (thus diminishing the total carbon content), but more especially at a white heat, thus favoring the formation of graphite during slow cooling. It increases the fusibility and fluidity of iron; it lessens the formation of blow-holes; by reducing iron oxide it apparently removes one cause of red-shortness; it hinders at high temperatures the oxidation of iron, and probably of the elements combined with it. Silicon steels with 1 to 2 or even 2.5 per cent silicon, sometimes excellent for cutting hard steel, have been made. Iron absorbs silicon greedily, uniting with it in all proportions at least up to 30 per cent, and apparently the more readily the higher the temperature, absorbing it even at a red heat when imbedded in sand and charcoal. Though silica can neither be reduced by iron alone nor by carbon alone, it is readily reduced by carbon if iron be present to alloy with the resulting silicon.

"Silicon may be oxidized by both carbonic acid and carbonic oxide: it is removed from molten iron very rapidly by atmospheric air, and by simple contact with iron oxide, magnesia, and other bases." *

* Howe, vol. I. p. 36.

117. Influence of Silicon on Physical Properties.—The effect of silicon is to increase the strength and to reduce the ductility of steel, as shown in

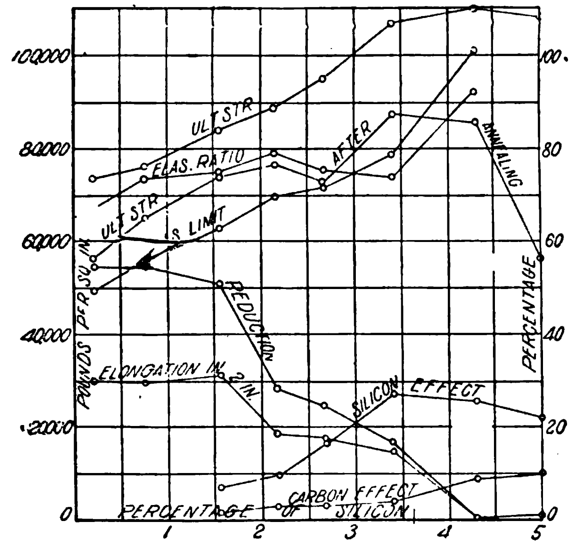

FIG. 74.—Physical Properties of Silicon Steel, showing the Effects of Carbon and Silicon. (Hadfield in *Jour. Ir. & St. Inst.*, vol. II. p. 222.)

Fig. 74. It also has a decided effect in increasing the soundness of ingots and other castings, thus preventing blow-holes, and by reducing the iron oxide it to that extent prevents red-shortness.

118. Effects on Cast Iron.—The effect of silicon on cast iron is to increase its fluidity, and to change the carbon to the graphitic form. Thus hard white cast iron is reduced to soft gray iron by the addition of silicon, while its tensile strength and its ductility or toughness is increased.

INFLUENCE OF MANGANESE ON IRON AND STEEL.

119. In General.—"Manganese alloys with iron in all ratios, being reduced from its oxides by carbon at a white heat, and the more readily the more metallic iron is present to combine with it. It is easily removed from iron by oxidation, being oxidized even by silica, and partly in this way, partly in others, it restrains the oxidation of the iron, while sometimes restraining, sometimes permitting, the oxidation of the other elements combined with it. It is also apparently removed from iron by volatilization. Its presence increases the power of carbon to combine with iron at very high temperatures (say 1400° C.), and restrains its separation as graphite at lower ones.* By preventing ebullition during solidification and the formation of

* Prof. J. O. Arnold says that with 1 per cent manganese iron becomes saturated with 0.65 C, instead of 0.89 C with no manganese. Hence the softer qualities of Swedish steels of a given percentage of C, as they contain only about 0.25 per cent Mn. See Appendix B.

blow-holes; by reducing or removing oxide and silicate of iron; by bodily removing sulphur from cast iron and probably from steel; by counteracting the effects of the sulphur which remains, as well as of iron oxide, phosphorus, copper, silica and silicates, and perhaps in other ways,—it prevents hot-shortness, both red and yellow. (It does not, however, counteract the cold-shortness caused by phosphorus.) These effects are so valuable that it is to-day well-nigh indispensable, though admirable steel was made before its use was introduced.

"It is thought to increase hardness proper and fluidity, to raise the elastic limit and the ultimate strength, and, at least when present in considerable quantity, to diminish fusibility." *

120. Effect of Small Percentages of Manganese on Static Strength.—From over 400 tests of the strength of mild steel accompanied by chemical analyses, Mr. William R. Webster † estimates the effect of manganese in increasing the ultimate strength, as given in the following table:

TABLE XV.—INCREASE IN ULTIMATE STRENGTH FROM SMALL PERCENTAGES OF MANGANESE.

Manganese, Per Cent.		Increase in Ultimate Strength.	Total Increase in Ultimate Strength from 0 Manganese.
From	To	Lbs. per Sq. In.	Lbs. per Sq. In
0.00	0.15	3,600	3,600
0.15	0.20	1,200	4,800
0.20	0.25	1,100	5,900
0.25	0.30	1,000	6,900
0.30	0.35	900	7,800
0.35	0.40	800	8,600
0.40	0.45	700	9,300
0.45	0.50	600	9,900
0.50	0.55	500	10,400
0.55	0.60	500	10,900
0.60	0.65	500	11,400

121. Manganese-steel.—"While the small amounts of manganese in ordinary commercial steel increase its forgeableness, and within certain limits its brittleness, yet when so much manganese is present that its effects outweigh those of carbon, and thus forms a true manganese steel, the alloy becomes extraordinarily tough and difficultly forgeable: it possesses a combination of hardness and toughness which should be of value for tools which cut by impact, and which is not otherwise attainable, so far as I know, at least in any material available for the arts. Several attempts to utilize its remarkable properties have been made of late, and others are to be expected." ‡

"Briefly, manganese-steel of the best composition, with say 14 per cent

* Howe, vol. I. p. 42.

† See *Trans. Am. Inst. Mining Engineers*, vol. XXIII. p. 114.

‡ Howe, vol. I. p. 48.

of manganese and not more than 1 per cent of carbon, is very fluid; solidifies rapidly and with great contraction; does not form blow-holes, but pipes deeply; does not seem subject to segregation; is forgeable, but welds poorly if at all. Naturally brittle, only moderately strong, and with very low elastic limit, it is made extremely tough and very strong* and (under impact) stiff by quenching from whiteness, which neither cracks small bars of it, changes its fracture (which before forging is strongly crystalline), nor greatly raises its elastic limit; this, however, is greatly raised by cold stretching, only to fall on reheating. Test-bars stretch nearly uniformly, like brass, instead of necking like iron. It is so hard that it can barely be machined, but it is slightly softened by sudden cooling from very dull redness; is not brittle at blueness, nor (apparently) made brittle by blue-work, but is rapidly made brittle by cold-work, ductility being restored by reheating and quenching; does not recalesce † during cooling; its density (sp. gr. 7.83, for manganese 13.75), modulus of elasticity, and (apparently) its rate of corrosion are about the same as those of common iron; its electric resistance is enormous, thirty times that of copper and eight times that of wrought iron, but thrice as constant with varying temperature as that of iron; it can be magnetized very considerably temporarily, but only with most extreme difficulty, and hardly at all permanently." ‡

INFLUENCE OF SULPHUR ON IRON AND STEEL.

122. In General.—" Sulphur unites with iron probably in all proportions up to 53.3 per cent, being readily absorbed from many sources. It may, however, be prevented from combining with iron, and even expelled from it by many agents (e.g., basic slags, carbon, silicon, manganese, oxygen, water, ferric oxide). Certain of these in the blast-furnace prevent the sulphur present from combining with the cast iron, and in the conversion of pig iron into malleable iron, whether by puddling, by pig-washing, or by the basic process, much of the sulphur of the cast iron is expelled. It causes cast iron to retain its carbon in the combined state. Carbon and sulphur and perhaps also silicon and sulphur are mutually exclusive within limits. Sulphur makes malleable iron red-short and interferes with its welding, but these effects are largely effaced by the presence of manganese. It is thought to make cast iron harder, though this effect is at least in part due to its causing it to retain the carbon in the combined state. It increases the fusibility of cast iron, but makes it thick and sluggish when molten, and gives rise to blow-holes during its solidification." §

123. Red-shortness.—" Sulphur has the specific effect of making iron exceedingly brittle at a red heat, and of destroying its welding power. Its

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* Tensile strength raised from 80,000 to 100,000 lbs. per square inch, and elongation in 8 inches raised from two per cent to *forty-five per cent!*

† See Art. 130, (*c*), for definition of this term.

‡ Howe, vol. I. p. 361.

§ Howe, p. 48.

effect are in general most marked at a dull-red heat, and irons which crack at this temperature owing to the presence of a small percentage of sulphur, may often be readily forged at higher temperatures, while when cold they are as malleable and indeed often more malleable, than non-sulphurous irons. If, however, the percentage of sulphur is considerable, the iron is no longer malleable even at temperatures above redness. The red-shortness imparted by a given percentage of sulphur is probably independent of the percentage of carbon which accompanies it; but more sulphur can usually be tolerated in steel rich in carbon than in others, because such steel usually contains much manganese also.

"The rail-steel of our Eastern mills has usually from 0.03 to 0.06 per cent sulphur; that of our Western mills has usually somewhat more, occasionally as much as 0.10 or 0.12 per cent, and even exceptionally 0.14 per cent. When sulphur is under 0.08 per cent its effects [on red-shortness] are probably almost completely effaced by the presence of 0.80 per cent manganese, since with this composition the red-shortness is so slight that T rails, the formation of whose thin flanges necessitates great malleablness, can be rolled with so little cracking that at some mills only 0.4 per cent of the rails made are of second quality (i.e., have cracked flanges).

"Pieces of a shape which can be produced without necessitating such extreme malleableness as the formation of the thin flanges of T rails requires may contain more sulphur; but it is rare to find more than 0.12 sulphur in any steel. Crucible tool-steel has ordinarily less than 0.01 per cent. Nail-plate has usually from 0.05 to 0.10, boiler-plate from 0.02 to 0.08, per cent.

"Manganese counteracts the effects of sulphur. In many cases 4.5 parts by weight of manganese so far counteract the effects of one part of sulphur as to permit the rolling of flange T rails." * See also Appendix B.

124. Tensile Strength and Ductility.—The effect of sulphur on the tensile strength and ductility of iron and steel has formerly been somewhat in doubt. It was known, of course, that by producing red-shortness it may indirectly cause weakness of the cold specimen from external or internal cracks resulting from the red-shortness in the process of rolling. It has now been shown by Messrs. Andrews and Arnold, that *as small an amount of sulphur as* 0.05 *per cent, in the form of sulphide of iron, may form in thin meshes, and so very greatly reduce the strength and toughness of steel.* Manganese reduces but silicon magnifies this action. Annealing causes these sulphide flakes to collect in masses, thus largely destroying its weakening effects.† This new and important discovery will serve to explain some of the many astonishing failures of steel which have hitherto been entirely unintelligible. (See Appendix B.)

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* Howe, pp. 52 and 53.

† Prof. J. O. Arnold in *Trans. Inst. Civ. Engrs.*, vol. CXXIII, 1896, p. 209; and Mr. Thos. Andrews in *Engineering*, Jan. 17, 1896.

INFLUENCE OF PHOSPHORUS ON IRON AND STEEL.

125. In General.—" Phosphorus, the steel-maker's bane, unites with iron probably in all proportions at least up to 26 per cent, being readily absorbed by it, especially at high temperatures and when under deoxidizing conditions, from acid phosphates and silico-phosphates. Fortunately it is readily removed from iron, especially under strongly oxidizing conditions, by contact with strong bases (oxides of iron and manganese, the alkalies and alkaline earths) and by basic silicates and even silico-phosphates, by alkaline carbonates and nitrates, and by fluor-spar. It is volatilized under many conditions, e.g., when phosphates are heated with carbon (the presence of metallic iron more or less completely prevents this volatilization), and when molten phosphoric cast iron is brought in contact with alkaline matter or (probably) with fluor-spar. In the blast-furnace, however, phosphorus is not effectively volatilized, for any which volatilizes immediately recondenses. Hence in the blast-furnace nearly all the phosphorus passes into the metal, though a little is found in the slag if the deoxidizing conditions be weak. In puddling 90 per cent, and in the basic Bessemer process 96 to 99 per cent, or even more, of the phosphorus initially present may be removed under favorable conditions.

Phosphorus increases the static strength of low-carbon iron and steel, but it greatly reduces its resistance to shock. It increases the elastic limit, but reduces the ultimate elongation and contraction. Carbon greatly intensifies the bad effects of phosphorus, and silicon may intensify them, but certainly to a very much smaller degree if at all. " Rapid cooling and forging during cooling, by preventing the coarse crystallization to which phosphoric iron strongly inclines, oppose the effects of phosphorus on ductility. It is certain that phosphorus does not always diminish the hot-malleableness of iron, at least at moderate temperatures; but by increasing the tendency to coarse crystallization it probably diminishes malleableness at very high temperatures, and especially when the iron has slowly cooled without forging from a very high temperature to a somewhat lower though still high one, as this seems to be the condition most favorable to coarse crystallization." *

126. The Condition of Phosphorus in Iron.—" In ingot metal phosphorus exists chiefly if not exclusively as phosphide; but in weld metal it probably exists both as phosphide and as phosphate, i.e., as part of the mechanically intermixed slag, in which condition it is reasonable to suppose that its effect on the mechanical properties of the metal should be comparatively slight. Many and perhaps an indefinite number of phosphides of indeterminate composition may exist in iron, for we find wide differences between the chemical behavior of different portions of phosphorus, even in one and the same piece of iron, and apparently equally wide discrepancies between the effect of a given quantity of phosphorus on the physical properties of different irons.

* Howe, p. 54.

The differences in the chemical behavior of phosphorus are exemplified by the fact that, on dissolving some steels in chlorhydric acid, part of the phosphorus escapes as phosphoretted hydrogen, part is found as phosphoric acid, part apparently as some lower oxygen acid, while still another part is insoluble.

"The existence in solid iron of a definite phosphide of iron, Fe_3P, and probably that of a definite phosphide of manganese, Mn_2P_3, is well established." *

127. Effect of Phosphorus on the Ductility of Soft Steel.—While phosphorus seems to increase the strength of low-carbon steel, it very much diminishes its ductility, and this is now regarded by engineers as a very

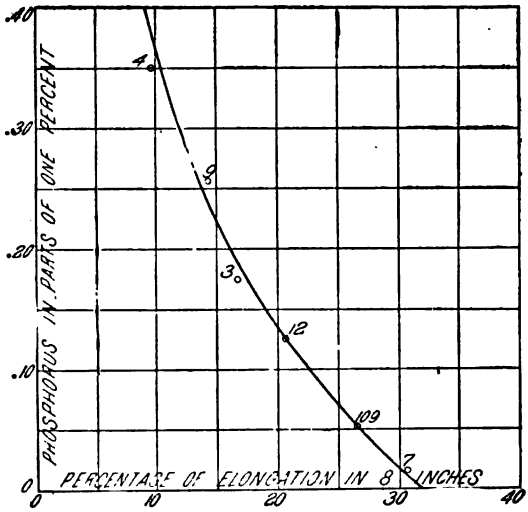

FIG. 75.—Effect of Phosphorus on the Ductility of 0.10% to 0.20% Carbon Steel. Numbers indicate No. of Observations averaged. (Howe's *Steel*, p. 68.)

dangerous ingredient, and its maximum percentage is carefully specified in the better grades of structural steel. Fig. 75 illustrates this effect on steel having from one tenth to one fifth of one per cent carbon (0.10 to 0.20) and a tensile strength of from 55,000 to 64,000 lbs. per square inch, when the phosphorus ingredient is less than one tenth of one per cent. The locus drawn on this diagram is the most probable curve, showing the law of decrease in ductility for increase in phosphorus for the 144 tests here plotted. Each plotted point represents the average of the number of tests indicated in the attached numerals. When the phosphorus ingredient reaches one fourth of one per cent the tensile strength of the same steel is upwards of 70,000 lbs. per square inch. The diagram in this figure shows a loss of ductility represented by a diminished elongation in a length of eight inches from 30 per cent down to 10 per cent, as the phosphorus ingredient rose from two one-hundredths to thirty-five one-hundredths of one per cent.

* Howe, p. 55.

While this diminution of the percentage of elongation for increasing percentages of phosphorus is a strong proof of increased brittleness, various impact tests on high-phosphorus steel, and surprising and remarkable accidents with such steel, lead to the conclusion that the brittleness of high-phosphorus steel under suddenly applied loads and under shock is even greater than would be indicated by the diagram in Fig. 75. The maximum proportion of phosphorus now (1896) allowed under the better specifications for structural steel is from four one-hundredths to eight one-hundredths of one per cent. This requirement is readily complied with by the basic open-hearth process.

128. Effect of Phosphorus on Static Strength.—From over 400 determinations of strength with corresponding chemical analyses, Mr. William R. Webster has shown* that phosphorus adds to the static strength of low-carbon steel approximately as indicated in the following table:

TABLE XVI.—EFFECT OF PHOSPHORUS ON STATIC STRENGTH.

For Carbon, Hundredths Per Cent.	Increase of Ultimate Strength per 0.01 Per Cent P added.	Effect of Unit of P to Unit of C as 1 to—
9	900	$1\frac{1}{8}$
10	1000	$1\frac{1}{4}$
11	1100	$1\frac{3}{8}$
12	1200	$1\frac{1}{2}$
13	1800	$1\frac{5}{8}$
14	1400	$1\frac{3}{4}$
15	1500	$1\frac{7}{8}$
16	1500	$1\frac{7}{8}$
17	1500	$1\frac{7}{8}$

129. Limiting Values of Chemical Constituents Allowable.—Since nearly all the constituents of iron and steel are more or less injurious, it is well to specify the upper limits which will be allowed in a given product. These upper limits should also be placed as low as possible without increasing appreciably the cost. Metcalf † gives, in his work on *Steel* (1896), the following as such a set of limits:

"Silicon .... $< .10$ of one per cent.
Phosphorus .... $< .05$ ‡
Sulphur .... $< .02$
Manganese .... $< .50$, or even $< .30$
Copper .... $< .03$
Carbon to meet the physical requirements."

* See *Trans. Am. Inst. Mining Engineers*, vol. XXIII. p. 114.

† Wm. Metcalf, past President Am. Soc. Civ. Engrs., who has spent his life in manufacturing steel, and hence whose judgment in such matters can be relied on.

‡ The specifications put out in 1896 by the Association of American Steel Manufacturers contain limitations of the phosphorus ingredient of 0.04 for rivet and fire-box steel; 0.06 for flange or boiler steel; of 0.08 for railway-bridge steel; and of 0.10 for steel for buildings and highway bridges. (See Appendix D.)

The following chemical requirements were adopted by the Illinois Steel Co. for steel plates in 1895:

Quality.	Carbon.	Manganese.	Sulphur.	Phosphorus.
Fire-box............	.16	.35 to .50	Not over .040	Not over .020
Boiler.............	18	.35 to .60	" " .045	" " .040
Flange	.18	.35 to .60	" " .045	" " .040
Ship...............	.15	.35 to .65	" " .060	" " .080
Tank..............	.10	.40	" " .100	" " .120

HARDENING, TEMPERING, AND ANNEALING.

130. Heat Changes in Carbon Steel.—When steel contains from 0.50 to 1.00 per cent carbon it crystallizes in a number of ways, and the ingredients arrange themselves in a number of different chemical combinations at different temperatures. The changes in the structures of steel when passing through the critical temperatures are discussed in Art. 107. Thus there is a critical temperature between a cherry-red and a low-yellow heat (about 700° C. or 1300° F.), at which the state of the carbon changes from cement to combined (or hardening) carbon as the temperature slowly rises past this point, and from hardening to cement again as the temperature slowly falls below it. Thus at temperatures above 700° C. the carbon is all in the hardening state, and the crystallization is that which corresponds to this chemical combination. When the temperature slowly falls below this limit, however, the carbon is expelled from its former associations, the metal arranges itself in a new series of crystals, and this state of transition is marked by many peculiar phenomena.

(*a*) At this time, whether the passage through this transforming region be upward or downward, the metal shows great weakness. The molecules seem to largely loose their coherence, and the bar bends readily, or the metal flows freely under a comparatively low stress. After passing this stage, in either direction, the strength is greatly increased.

(*b*) When this stage is reached with a rising temperature, or when the cement carbon is changing to the combined or hardening state, a great deal of heat is consumed to effect this change, so that notwithstanding the continued absorption of heat the temperature ceases to rise for a time. That is to say, the heat added here becomes latent, or does work in effecting the change in the carbon condition and the new arrangement of the crystals.

(*c*) When this stage is reached with a falling temperature, the carbon separates itself from its former chemical union with the iron, and forms a new compound (Fe_3C, carbide of iron), and the carbon is now said to be in the cement state. This involves a new crystalline arrangement also, a temporary weakening of the metal, and a giving out of the latent heat as sensible heat. That is to say, as the bar cools down past this point its temperature

suddenly increases, though in a cooling atmosphere, from the transformation of the latent to sensible heat. This action is called *recalescence.*

(*d*) As a result of this increase in temperature the contraction is changed to an expansion, to be followed by a contraction when the temperature begins falling again.

131. Hardening of Steel.—In order to obtain a hardened steel it is necessary to retain the carbon in the hardening state (chemically combined with the iron in the ratio of about 99 Fe to 1 C) when cold. Although the carbon always is in this state at high temperatures (above about 1300° F.), yet it will always change from this to the cement state in falling through this critical temperature, if any appreciable length of time is given it to effect the corresponding chemical and structural changes. It follows, therefore, that *hardening consists in cooling steel rapidly from a temperature above a low-yellow heat*, in order that it shall not have time to effect these changes. Doubtless a portion of this change does occur, but with very sudden cooling in a liquid bath most of the carbon is retained in the hardening form. The degree of suddenness of cooling depends on the kind of liquid used; hence the great variety of cooling baths in use, such as mercury, salt-water, fresh-water, oil, tallow, tar, etc. These liquids cool the steel with a relative rapidity in the order here named, mercury cooling it most rapidly.

132. Tempering of Steel.*—Having cooled a piece of steel suddenly, and so retained its carbon in the hardening state, it is usually found to be too hard and brittle for the mechanical uses to which it is to be put. It must now be softened, or tempered, by heating it up to the proper temperature, and cooling slowly, which will suffice to change a certain portion of the carbon into the cement, or carbide, form. Evidently the higher this temperature the more complete will be this change. Even though in tempering the heat should reach the critical low-yellow stage, where the carbon takes on the hardening form, if it be followed by slow cooling, as in the air, it will change back to the cement state, and the piece will be entirely softened, or annealed. When the reheating is well below this critical temperature, and followed by slow cooling, the piece will be softened in proportion to the temperature reached and to the time during which it was kept at such temperature. Thus any particular degree of hardness, or temper, can be obtained by intelligent and skilful handling.

133. Effects of Hardening and Tempering.—The effect of hardening, that is, of sudden cooling of steel containing 0.50 per cent carbon or more, is to retain the carbon mostly in the hardening state, and to give a degree of hardness proportioned to the percentage of carbon and the suddenness of the cooling. Not only is the product harder, but its strength and its elastic

* The word "temper" is used in two senses. The steel-maker uses it to indicate initial hardness, as produced by the percentage of carbon, as low, medium, or high temper. The user of steel uses this term to indicate final hardness, as determined by the heat (color) to which hardened steel was reheated, as straw, brown, blue, etc. (See table in Art. 107.)

limits under all kinds of stress is greatly increased. The ductility, however, is reduced as the strength is increased. Thus steel containing 0.50 per cent C, which cooled in the air after rolling at a red heat, having a normal tensile strength of 67,000 lbs., an elastic limit of 34,000 lbs. per square inch, and an elongation in eight inches of 25 per cent, when heated to a low-yellow heat and quenched in water will have its tensile strength raised to 150,000 lbs. per square inch, an increase of 73 per cent; its elastic limit raised to 68,000 lbs. per square inch, an increase of 100 per cent; and its elongation reduced to 2.5 per cent, a loss of 90 per cent. Such a steel can now be tempered and brought to any condition intermediate between these limits. When hardened in oil all the above effects are less marked.

134. Annealing consists in heating to or slightly above the critical point described in the preceding articles, that is, to a medium average color (or 655° C. or 1150° F.), and cooling slowly and uniformly. This removes all the hardening effects of a previous rapid cooling, and it also removes all the internal stresses produced by a previous unequal heating and cooling, as when portions of the plate have been heated for forging, and also the effects of such hot or cold working, as rolling and hammering when hot, or punching, shearing, bending, pulling, crushing, hammering, twisting, rolling, etc., when cold. That is to say, proper annealing restores the metal to its normal condition. In the case of steel, it changes all the carbon to the cement or non-hardening condition, and at the same time relieves all internal stresses. To insure its full effects, however, the cooling should be slow and uniform through the entire mass of the body. It should not, however, be left to cool down with the furnace, as this holds it too long at the high temperature. It should be removed from the furnace as soon as heated through, but may then be covered with quick-lime or powdered charcoal, to insure a slow and even cooling. Merely heating to the required temperature and cooling in the open air, possibly in contact with cold metal surfaces, and exposed to draughts, does not satisfy the necessary conditions of proper annealing, since the cooling is too rapid and is wanting in uniformity. The heating should be in an oven large enough to take the entire body, and not by a forge, or by a fire built over the body, and the rate of heating should not be too rapid.

Wire is usually annealed in cylindrical pits built of fire-brick, and covered over, the fire passing around them. This, and all other processes of annealing in which the steel is exposed to the air, causes an oxide scale to form on the exposed surfaces, which scale has then so strong an affinity for the carbon of the underlying steel, that it decarbonizes it to a very slight depth below the scale. While this is of no consequence in large masses, as in structural forms or in billets, it is quite fatal in such cases as fine spring-wire, or wire to be made into drills, punches, graving-tools, and the like, as this decarbonized surface cannot be hardened. To prevent this action the annealing-pots are commonly filled with charcoal, which serves both to exclude most of the air and to deoxidize what is left, but still some oxidation

of the steel surfaces occur, with a corresponding decarburization beneath it.* This process also fails to give perfect results, and some other means must be found.

The Jones method (patented) consists in putting material in a closed tube from which the air is all expelled by some other non-oxidizing gas, and then placed in the furnace, and turned occasionally, the gas constantly flowing through the pipe. This seems to be a perfect method, the surfaces remaining absolutely bright and untarnished.

Metcalf uses a closed pipe also, with a loose cap, with resin thrown into the extreme end, which, by volatilizing on first entering the furnace, drives nearly all the air out of the tube. While this method leaves the surface slightly tarnished it prevents all decarburization of the steel.

CORROSION.

135. Corrosion of Iron and Steel.—Iron is corroded by the combined action of oxygen and water or carbonic acid and water. Neither of these elements acting alone will start corrosion on iron. Iron will remain bright indefinitely in dry air, or in water free from oxygen and carbonic acid. Acid fumes, sulphuretted hydrogen, chlorine, etc., will start corrosion without the presence of water. After a rust coating has once formed, however, it will progress in dry air. Corrosion proceeds more rapidly when the surface is alternately wet and dry, or when the moisture coating is very thin, than when deeply immersed.

While cast iron resists corrosion better than wrought iron and rolled steel, when all these have their natural surfaces unbroken, yet if all be dressed, and the bright surfaces exposed, cast iron corrodes more rapidly than the rolled metal. No relation has been established between the chemical composition of iron and steel and the rate of corrosion. Neither can it be affirmed that wrought iron or steel corrodes the more readily. (See Howe's Met. of Steel, §§ 160–169.)

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* Metcalf says it is very common to maintain the heat too long in using this and other methods of annealing, thus spoiling vast quantities of good steel every year. Some of the carbon changes to the graphitic form when the heat is too long maintained.—*Steel*, p. 88.

CHAPTER X.

THE MINOR OR AUXILIARY METALS OF CONSTRUCTION AND THEIR ALLOYS.

THE MINOR METALS.

136. Copper.—Copper, being found native, has been used in the arts, both alone and alloyed with tin and zinc, from the earliest times. It is so commonly used now for electric conductors that its more important qualities are well known. Its specific gravity is from 8.6 in castings to 8.9 in rolled and drawn forms, giving thus an average weight of 550 lbs. per cubic foot. It melts at about 2000° F., volatilizes at a white heat, and when cold does not oxidize in dry air, but does in a moist or acid atmosphere. It unites with oxygen at a red heat, forming both the black and the red oxides, the latter of which is soluble in melted copper, and makes it brittle when cold. Commercial copper is never pure,* the ordinary ingredients being iron, arsenic, antimony, and the red (cuprous) oxide. This last can be removed by melting the copper with charcoal and stirring with a stick of green wood, this process being called "poling."

Cast copper has a tensile strength of some 25,000 lbs. per square inch, with a very low elastic limit, of some 8000 lbs. When rolled, or drawn into wire, its strength may be raised to 50,000 or 60,000 lbs. per square inch, depending on the amount of work done upon it. It is then "hard-rolled" or "hard-drawn," and it has very little ductility. Its elastic limit is then very nearly equal to its ultimate strength. By heating it a bright cherry-red and cooling it either slowly or quickly, it becomes softened again, or annealed.

137. Zinc, which is commonly called "spelter" when cast, is a hard, brittle, white metal, with a highly crystalline fracture. It becomes malleable and ductile at about 200° to 300° F., but is brittle again at higher temperatures. Its specific gravity is 6.9 cast and 7.1 rolled. It melts at 800° F., and volatilizes at about 1900° F. It rapidly oxidizes in air at a red heat, and at a bright red heat, at which copper melts, zinc distils. It is mostly used as an alloy in brass, German silver, etc., and as a coating to iron and steel sheets and wire, which process is called galvanizing. It is a common

* The Lake Superior coppers are among the purest in the world.

electropositive element in electric batteries. Its common impurities are iron, lead, and arsenic.

138. Tin.—Tin is a white, lustrous, and extremely malleable metal, as is evidenced by its form in tin-foil. Its specific gravity is 7.3; it melts at 450° F., but does not readily volatilize. Commercial tin contains various portions of many elements such as lead, iron, copper, arsenic, antimony, bismuth, tungsten, and sometimes manganese and zinc. It is used for coating iron plates, and to alloy with copper and zinc. Its low melting-point causes it to be used for safety-plugs in boilers, as its melting-point corresponds to a steam-pressure of about 400 lbs. per square inch above atmospheric.

139. Aluminum is a white, soft, malleable metal of extreme lightness, its specific gravity being only 2.56 when cast and 2.75 when rolled. It melts at about 1150° F., but does not volatilize at ordinary melting temperatures. It is especially free from oxidation and corrosion in air, as neither oxygen, carbonic acid, carbonic oxide, sulphuric or nitric acid, sea-water, nor sulphuretted hydrogen has much effect on it. It is, however, readily dissolved by hydrochloric acid and by caustic alkalies. Its strength pure, when cast, is only about 18,000 lbs. per square inch, with low elastic limits in tension and compression. When rolled or drawn into wire its strength is raised to from 25,000 to 50,000 lbs. per square inch with elastic limits of about one half the ultimate strength. It is seldom used in a pure state because of its softness, but makes with copper, iron, zinc, and tin remarkably strong and malleable alloys, which will be discussed as aluminum alloys.

Aluminum may be rolled either hot or cold. It is annealed by bringing it to a low red heat and cooling slowly. In casting aluminum care must be taken to provide for the great shrinkage. It is best to cast in hot iron moulds and to cool from the bottom artificially, keeping melted metal supplied at the gate to supply the shrinkage. Casting under pressure also gives good results.

It is difficult to obtain aluminum in a perfectly pure state, and very slight amounts of impurities largely affect its properties. The common impurities are iron and silicon. It is now (1896) supplied regularly by the Pittsburg Reduction Company, under a guarantee of 98 % pure at 50 to 55 cents a pound, and will be furnished 99% and 99.6% pure at special rates.

THE ALLOYS.

140. Nature of Metallic Alloys.—Any permanent mixture of two or more metals is termed an *alloy.** Neither the appearance nor the mechanical properties of an alloy can be predicated upon those of the constituent metals, and the surprising character of the results produced by various mixtures has led to an enormous number of specially named products, each possessing certain desirable qualities, the ingredients usually being, for a time at least,

* When mercury is one of the constituent metals the product is termed an *amalgam*.

trade secrets. Between 1875 and 1880 the U. S. Test Board made so thorough an examination of all possible mixtures of the more usual ingredients found in alloys (copper, zinc, and tin), that the proprietary or trade names formerly used exclusively for these products are now giving place to stated percentages of the constituent metals.

In a general way, mixtures composed almost exclusively of copper and zinc are termed *brass*, while those composed mostly of copper and tin are called *bronze*, while compositions of all three of these elements are called *composition metal*, or perhaps also *bronze*. All these terms are used loosely, however.

An alloy, though ever so uniformly mixed when in a melted state, is usually a conglomerate mixture, after cooling, analogous with granite. Some pure chemical unions are formed, and certain substances may crystallize out, leaving the more fusible solution or mechanical mixture to form the matrix for the entire mass when cold. In most cases there is a decided tendency for the metals to separate before cooling, especially when they are of different specific gravities, this separating action being called *liquation*. To prevent this the mixture is stirred vigorously just before pouring, which is done at as low a temperature as possible. The quicker the metal cools in the mould, also, the better, so that it is common to cast alloys in iron moulds in order to chill the metal, or to cool it suddenly. To obtain constant mechanical qualities in any given alloy seems to be almost a practical impossibility. To secure even approximately uniform results requires more care and expert superintendence than the manipulation of any single metal. The greater the number of the constituent metals, also, the greater are the difficulties encountered. Manufacturers should be slow, therefore, to contract for alloys having definite mechanical qualities of a high order, if they have not had a considerable experience in meeting with similar demands.* Almost as much seems to depend on the manipulation as on the metals and proportions employed; but this subject is too large to be entered upon here.†

141. The Copper, Zinc, Tin Alloys.—In a general way, some alloy of two or more of these metals, copper being always one, is used for all purposes where strength, hardness, or malleability is desired in a non-corrosive metal. In other words, zinc and tin, one or both, are added to copper to harden and strengthen it. Formerly an alloy was used also for large cast guns (then called gun-metal), but these are now made of hollow-forged steel. As shown by Fig. 76, the valuable alloys are those in which copper forms the controlling element. This diagram is based on that principle in geometry which makes the sum of the normals from any point on the interior of an equilateral triangle equal to the altitude of the triangle. If the three altitudes be each taken as a scale of equal parts on which are indicated proportions (percentages) of copper, zinc, and tin respectively, these ranging from zero to

* The author has known of many failures of contractors in this field.

† See *Mixed Metals*, by Prof. Hiorns, 1890, Macmillan & Co.

100, then to the same scale the sum of the three normals from any point in the triangle will be 100, and hence these three normals may be used to indicate the percentages of the three metals which unite to form that alloy which is represented by that point in the triangle.* An alloy of any two of these finds its place along one side of the triangle, of which the three apices make the 100-per-cent ends of the three metal scales. A little study of Fig. 76 will make this clear.

The contour-lines on this figure were drawn by the author after plotting on this triangle the *tensile strengths of cast bronze of known composition*

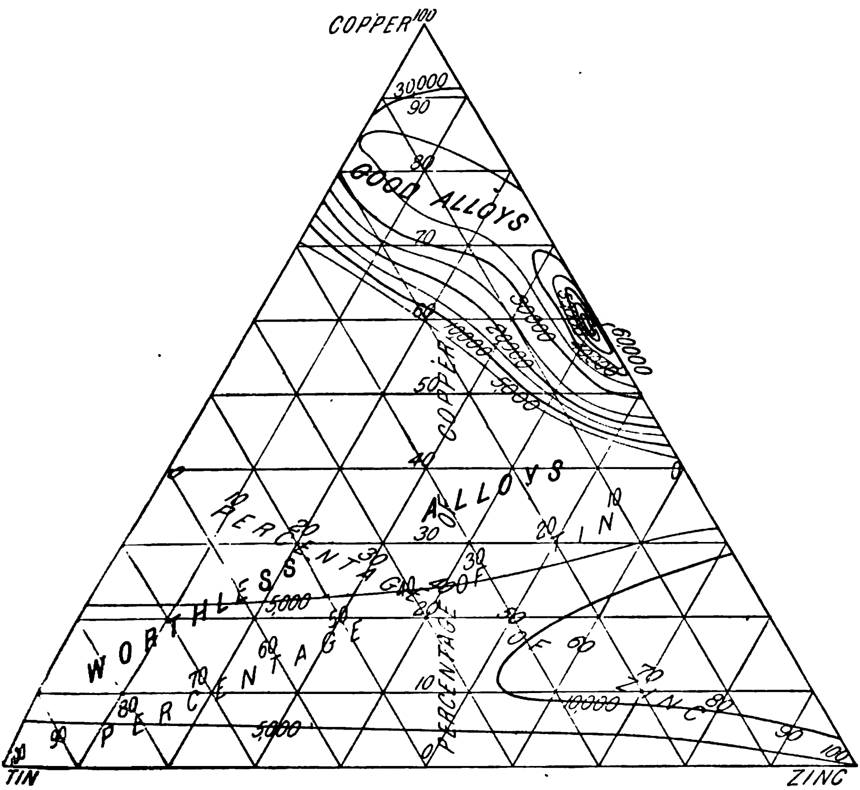

FIG. 76.—Showing the Tensile Strength of the Cast Copper-Tin-Zinc Alloys of all Possible Mixtures. (Plotted by the author from the results of tensile tests reported by the U. S. Test Board in 1881.)

from all reliable sources. Dr. Thurston's chart, after which this is modelled, was made from torsion tests by using a constant factor to reduce to equivalent tensile strength. The author finds the tension tests themselves do not agree very well with the values given on that chart, and hence he has drawn

* This method of representing these triple alloys was first used by Dr. R. H. Thurston, *Trans. Am. Soc. C. E.*, 1881.

a chart from the tension tests themselves. It must be understood, however, that, as stated in the previous article, so much depends on the purity of the ingredients, and on the manipulation of the process of melting and casting, that this chart, or any similar record, must be taken as showing what *may* be obtained rather than as what *will* be obtained from the use of these particular mixtures.

THE BRASSES.

142. The Brasses—Copper and Zinc.—The most valuable brass alloys contain from 65 to 85 per cent of copper and 35 to 15 per cent of zinc (3 to 5 of copper to 1 of zinc). These mixtures are all strong and ductile, not too hard to be readily worked in the lathe (a little tin, say 2 per cent, helps it for this purpose), are readily rolled into plates and drawn into wire, and under various names are the brasses of commerce. The French standard mixture for sheet brass is 67 Cu to 33 Zn (2 Cu to 1 Zn), and care is taken to use only the purest metal for this purpose, as a very slight amount of iron or silicon (or lead in case of wire-drawing) greatly lessens its ductility. Rolled or hammered brass is annealed by heating to a cherry-red and cooling either slowly or rapidly.

40 *Muntz-metal* and *Sterro-metal* are used for ship-coverings in place of copper. The former contains ~~33~~ per cent of zinc, the large proportion of zinc producing a corroded surface which prevents the attachment of barnacles. The latter contains, in addition, 1.5 to 2 per cent of iron, which greatly strengthens it. It is also used for hydraulic cylinders carrying very great pressures.

Brass Castings should contain some tin when used for bearings, as this increases the hardness. Two or three per cent is sufficient. One or two per cent of lead increases its adaptation to turning, filing, and polishing, while from 1 to 6 per cent aluminum adds greatly to its strength and ductility.

In all cases, when melting copper, brass, or bronze, great care must be exercised to keep the air from the metal, in order to prevent oxidation. This is done by covering the metal, in the crucible, with a thick layer of powdered charcoal. The copper is first melted alone, in a deoxidized flame, and then the scrap brass and zinc (previously melted, these fusing at a much lower temperature) are added and the whole stirred vigorously to effect a thorough mixing. Sometimes this mixing is done after the crucible is removed from the furnace. If it is done in the furnace, the dampers should be nearly closed to prevent an excessive heat, which would vaporize the zinc.

A new brass-melting furnace is shown in Fig. 77,* in which crucibles are not used. The metal is charged at the upper door upon a sloping hearth from which it falls, when melted, upon the hearth proper, from which it is drawn from the tap-hole as shown. The flame from the adjacent fire can

* Designed and built by J. W. Bennett & Co., Pittsburg. From *Engr. News*, Oct. 1, 1896.

be turned into either of these chambers by the opening of suitable dampers, or into both at once, or stopped off entirely by the damper at the top of the flue. A furnace having 600 to 800 lbs. bosh capacity (5000 to 6000 lbs. per day) occupies a space of only 30 to 40 sq. ft. By means of the top damper the character of the flame may be so controlled as to prevent excessive oxidation.

If iron moulds are used, they should be heated and the interior surfaces

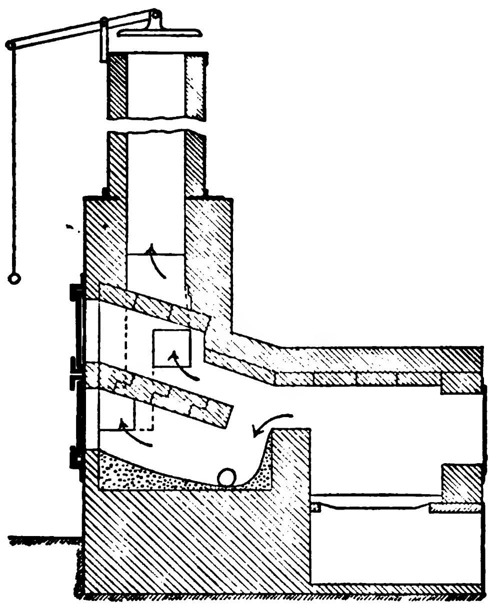

FIG. 77.—A New Brass-melting Furnace.

coated with a mixture of resin (3 pts.) and lard-oil (1 pt.) to prevent adhesion. In pouring, the metal must be very carefully skimmed. The pattern should be made to allow a shrinkage of ¼ in. per foot. For common castings green sand is used, but for fine work the moulds are dried.

143. Delta-metal, which is an improvement on sterro-metal, is a proprietary composition, or brass, placed on the market since 1883 by a Mr. Alexander Dick (England), who used the Greek form of the initial letter of his own name to designate his product. His process consists in incorporating a *fixed* amount of iron by making first a saturated solution of iron (about 5 per cent) in molten zinc. To prevent all oxidation a little phosphorus is added to the melted copper. The proportions are varied for different purposes, having from 50 to 65 per cent copper, 50 to 30 per cent zinc, 0.1 to 5 per cent iron, and sometimes 0.1 to 1 per cent tin. This metal is as strong and ductile as mild steel, having a tensile strength, when rolled and annealed, of from 60,000 to 80,000 lbs. per square inch, with elongations in eight inches of from 40 to 14 per cent, respectively, at these limits.* When cast

* Tests made at Lloyd's Proving-house, as given by Hiorns.

in sand its tensile strength is 45,000 lbs., with an elongation of 10 per cent. It also resists corrosion perfectly.

144. Tobin Bronze is very similar to sterro-metal and delta-metal, the iron ingredient being somewhat less. Its composition is approximately 60 per cent copper, 38 per cent zinc, 1 to 2 per cent tin, with small portions (0.1 to 0.3 per cent) of iron and lead. Its remarkable properties are due to its rolling and annealing. As placed on the market,* its tensile strength is from 60,000 to 80,000 lbs. per square inch, with an elastic limit of 60 per cent of its ultimate strength, and an elongation of from 25 to 15 per cent in eight inches at these limits respectively. It may be regarded as having all the mechanical qualities of structural steel, with the advantage of being non-corrosive. It can be procured in sheets from $\frac{1}{16}$ inch to $1\frac{1}{2}$ inches thick, and in round rods from $\frac{1}{4}$ inch to 5 inches in diameter. It is readily forged at a cherry-red heat either by hand or by machinery. It also works well in the lathe. It seems, therefore, to be a practically perfect non-corrosive, engineering metal.

THE BRONZES.

145. The Bronzes—Copper and Tin.—Since tin is added to copper solely to harden it (it strengthens it very little), the copper-tin bronzes may be regarded as a kind of hardened copper. The ancients used this combination for their cutting-tools, and it is used largely at the present time to produce a very hard, non-corrosive metal, useful for many engineering purposes. If more than 25 per cent tin is used, the alloy, though hard, becomes very weak and brittle. The most common mixture is that of *gun-metal*, which consists of 90 per cent copper and 10 per cent tin. If more than 5 per cent tin is used, the metal loses most of its malleability when cold. With about 20 per cent tin the metal is very hard and sonorous, making it suitable for bells, gongs, and wind-instruments. The copper-tin alloys are annealed by *sudden* cooling, as by quenching in water from a red heat, while by slow cooling they are hardened. They differ in this respect from nearly all other metals.

By using 33 per cent tin (2 copper to 1 tin), a beautiful, hard, perfectly white alloy is produced, called *speculum-metal*, suitable for polishing for mirrors.†

146. Phosphor-bronze is a plain copper-tin alloy made by using a little phosphorus as a deoxidizer. It is also claimed that the phosphorus causes the tin to form a crystallized compound with the copper. It is mainly, however, as a cleanser of the melted metal from the oxide of copper that it is valuable. When used properly it forms a slag, and is skimmed off, and is not found in the finished product. The phosphorus is added in the form of phosphor-copper or phosphor-tin, these containing phosphides of copper

* By the Ansonia Brass and Copper Co., New York.

† Lord Ross's great telescopic reflector was made of this alloy.

or of tin. For a malleable product, to be rolled or drawn into wire, the tin should not exceed 4 or 5 per cent, and the phosphorus should not exceed $\frac{1}{10}$ of one per cent. For hard castings of great strength, as for pinions, valves, bearings, or bushings, use 7 to 9 per cent of tin and $\frac{1}{2}$ to 1 per cent of phosphorus. A greater amount of phosphorus, up to 4 per cent, increases the hardness and brittleness. More than 4 per cent phosphorus will make the product useless.

147. Silicon-bronze is now used extensively in Europe for electric conductors, as it has 70 per cent of the conductivity of copper, while phosphor-bronze has but 30 per cent, and steel 10.5 per cent. By using silico-bronze wires the poles may be put much farther apart than when copper wires are used.* While the proportion of silicon remaining in the alloy is very small, it has an excellent cleansing action, like phosphorus, without danger of developing brittleness. The "dose" of silicon, to be added to the melted copper or bronze, is prepared by the inventor, Weiller, as follows: "Take potassium silico-fluoride 450 parts by weight, powdered glass 600 parts, common salt 250 parts, carbonate of soda 75 parts, carbonate of lime 60 parts, and dried chloride of calcium 500 parts. Heat these in a covered plumbago crucible a little below the temperature where they begin to act on each other, when the whole is added to the melted copper or bronze, and vigorously stirred." The resulting slag is skimmed off.

148. Aluminum Bronze has now come to be regarded as one of the most valuable made. It is composed of from 5 to 12 per cent aluminum with from 95 to 88 per cent of copper. These alloys have remarkable ductility, combined with great strength. Thus the 5 to $7\frac{1}{2}$ per cent aluminum bronzes have, when rolled or forged, an ultimate tensile strength of from 70,000 to 80,000 lbs. per square inch, an elastic limit of over 40,000 lbs., and an elongation in 8 inches of over 30 per cent. With 10 per cent of aluminum, the rolled bars have an ultimate tensile strength of 100,000 lbs. per square inch, an elastic limit of 60,000 lbs., and an elongation of 10 per cent in 8 inches. If further rolled, it hardens and strengthens to 130,000 lbs. tensile strength with 5 per cent elongation. The 5 to 7 per cent bronzes can be hammered, rolled, and forged at a red heat, and are very similar in every way to mild steel. They are almost absolutely non-corrosive. It hardens by cold working, but may be annealed by heating to a red heat and quenching in water. It has a modulus of elasticity of about 18,000,000, which is higher than that of the alloys of copper, zinc, and tin.

On account of the excessive shrinkage of this alloy in hardening, it is necessary to provide a large sinking head in casting, and to so locate this as to supply to the cast form the necessary fluid metal to give a sound casting as it shrinks away in cooling.

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* The Austrian Railway Company puts its poles from 328 to 720 feet apart in open country when using silico-bronze wires.

149. Alloyed (or Hardened) Aluminum.—Just as a small percentage of aluminum added to copper greatly hardens and strengthens it, without destroying its ductility, so a small percentage of copper added to aluminum works a similar change in this soft metal. If more than 15 or 20 per cent of either be added to 85 to 80 per cent of the other, however, the resulting mixtures become hard, weak, and brittle, and entirely worthless as commercial products.

Both tin and zinc, up to 15 per cent, are used to harden and strengthen aluminum, while an alloy of 15 per cent zinc, 3 per cent tin, and 82 per cent aluminum is especially recommended. There are a number of secret mixtures of hardened aluminum, some of which are used for casting bicycle-frames.

150. Aluminum in Steel.—If about one pound of aluminum per ton of steel be added to the heat just before drawing or teeming, it prevents the formation and escape of gases, and gives solid ingots or castings. For steel castings two to three pounds per ton is now commonly added to the melted steel by throwing small pieces into the ladle as the steel is drawn from the furnace. In both methods it permeates the entire mass without artificial stirring, as manganese does, and seems to have very much the same effect. Its effect on cast iron are the same as those of silicon, but as it is much more expensive it is not used in this way. In steel, however, its use is common, as nothing seems to take its place. Besides preventing blow-holes it adds to the ductility of the product.

151. Alloys which Fuse below the Boiling-point.—The following remarkable alloys, all of which fuse at very low temperatures, may be used as safety-plugs in automatic fire-spraying pipe-systems in mills and for similar purposes.

TABLE XVII.—FUSIBLE ALLOYS.

Name.	Percentage of Ingredients.				Fusing Temperature.
	Bismuth.	Lead.	Tin.	Cadmium.	
Newton's	50	31	19	0	95° C.
Rose's	50	28	22	0	100° C.
Darcet's	50	25	25	0	93° C.
Wood's	50	24	14	12	66–71° C.
Lipoirtz's	50	27	13	10	60° C.

CHAPTER XI.

LIME, CEMENT, MORTAR, AND CONCRETE.

LIME AND NATURAL CEMENT.

152. Quick, or Fat, Lime.—If carbonate of lime ($CaCO_3$), as found in ordinary limestone, or marble, or chalk, be heated to a temperature of about 800° F., when it becomes a cherry-red, the carbon dioxide (CO_2) is driven off, and the oxide of calcium (CaO) remains, and is called quicklime. In a pure carbonate of lime 44 parts by weight of carbon dioxide (carbonic acid) are combined with 56 parts by weight of oxide of calcium, or quicklime.' Since the rock will contain some moisture, the amount of quicklime obtained from burning limestone will never be more than one half the weight of the stone charged. The calcium oxide, or quicklime, cannot be decomposed by heat, but it has a very strong affinity for water. When water is added to it, it rapidly rises in temperature, swells, and falls into an impalpable powder, and increases its volume to about three times its initial volume before the water was added. This process is called slacking, and the product is then called hydrated, or fat, lime (calcic hydrate), or slacked lime, or lime paste or putty when further diluted with water. The quicklime, or calcium oxide, will slack by absorbing moisture from the atmosphere, unless kept in closed vessels. It is therefore not kept in stock for any great length of time, as it becomes bulky and difficult to handle when slacked. It can be kept indefinitely without deterioration in the form of lime paste, or putty, if kept wet so as to exclude the air. Quicklime is not found, as such, in nature, since it has a tendency to recombine with carbonic acid from the atmosphere, and form carbonate of lime. Rocks composed of nearly pure carbonate of lime are found in all parts of the world, and they have been used in this way for the manufacture of quicklime for mortar from the most ancient times. In slacking, 18 parts by weight of water unite with 56 parts by weight of quicklime, making 74 parts of calcic hydrate, $Ca(OH)_2$. The heat generated in slacking greatly facilitates the process, and some limes will slack in boiling water which cannot be slacked by the use of cold water. Limes of this latter class are called "poor," in distinction from those which slack readily, which are commonly termed "fat."

153. Hardening of Lime-mortar.—When quicklime has been slacked and mixed with sand it forms what is commonly called lime-mortar, which

is used for laying brick and stone masonry, for plastering houses, and the like, where the mortar-joints will be exposed to the action of the air only. Because of the great shrinkage of lime-paste in drying, it cannot be used neat, but must always be mixed with several times its volume of sand. When exposed to atmospheric action, the hydrated lime, $Ca(OH)_2$, slowly unites with carbonic acid (CO_2), which is always present in the atmosphere, thus changing a portion of the hydrated lime back to its original form of carbonate of lime, leaving another portion in the hydrated form. Since the carbonic acid can have access to the lime only by the circulation of air through it, it follows that this chemical change occurs mostly at the outer and exposed surfaces of lime-mortar joints, and does not take effect at a distance from the surface to any appreciable extent, except through the lapse of long periods of time. In all cases, therefore, where it is necessary for the mortar to harden in a comparatively short time, lime-mortar must not be used.

154. Hydraulic Lime.—When a limestone contains from 10 to 20 per cent of clayey matter, new combinations of lime and the silica in the clay are formed in the furnace, if the temperature is sufficiently high, which causes the product to slack less readily, and with a much less increase of volume, than in the case of quicklime. Hydraulic lime is partially slacked on drawing from the kiln by adding from 15 to 20 per cent of its weight of water, and it is then thrown into large heaps. The steam thus formed causes it to slack in the course of a week, after which it is screened and packed for market. It cannot be kept in the form of paste, as fat lime always is, as it would harden, like cement. If this same rock be calcined at a high heat and reduced to a clinker but not fused, and then ground without slacking, it forms the natural cement described in the next article. It is changed from the one product to the other by the chemical reactions which occur at the higher temperature in the kiln. Mortar made with this lime will harden somewhat under water, by a process of partial crystallization, and hence it is called *hydraulic lime*. Limestones having a composition suitable to make hydraulic lime are very common in England and Europe, but are not common in America; hence what is there known as hydraulic lime is not known in America as an article of commerce.

155. Natural Cement.—Carbonate and magnesian limestone rocks containing from 20 to 40 per cent of clay, when calcined to a clinker, just short of fusion, and finely ground, give a product which sets or hardens quickly on the addition of about 25 per cent of its weight of water, without any increase of volume, and forms a permanent artificial stone which increases in strength and hardness for many years. This product is known as *natural cement*,* because it is produced wholly from a natural rock. It

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* In England and on the Continent this kind of cement is commonly called Roman cement, from a supposed similarity to the cement the Romans used on their hydraulic

has become customary to give to natural cements local geographical names, indicating the place of their manufacture. This is more especially appropriate since the natural cements made in a given locality will have the same general characteristics, because they are all made from the same sedimentary rock. These cements are very largely used in America, some of the principal varieties being the "Rosendale" cement, made near the Hudson River in Ulster County, N. Y., the "Utica" cement, made at Utica, Ill., the "Louisville" cement, made mostly on the Indiana side of the Ohio River in the vicinity of Louisville, Ky., and the "Milwaukee" cement, made at Milwaukee, Wis. Such cements are made at various other places in the United States and Canada, and are known by their corresponding local geographical names. These cements are now very cheap, and oftentimes are found to vary greatly in quality. While the better grades of natural cement are quite sufficient in strength for nearly all kinds of engineering works, the want of uniformity in their hardening properties is a serious objection to their use.

Some of the American natural cements are very quick setting, which is a further objection to them, since it is difficult to use the mortar or concrete made from them before it begins to set, or harden.

The old Roman cement used by the Romans in their hydraulic masonry constructions was made by mixing volcanic ashes with lime in proper proportions.

PORTLAND CEMENT.

156. Historical.—An artificial mixture of lime and clay in proper proportions, calcined to a clinker at a temperature of incipient fusion, and finely ground, is called *Portland cement*. It received this name in 1824 in England, where it was first made, from its similarity in appearance when hardened to the noted oolitic limestone from the "Isle of Portland"* long used in England for building purposes. It was patented in that year by Mr. Joseph Aspdin, a Leeds brickmaker, as an "artificial stone." He mixed pulverized limestone, taken from the public macadamized roads, with clay, by adding water enough to reduce it to a liquid form. This was then dried and burned "in a furnace similar to a lime-kiln till the carbonic acid is entirely expelled." The necessity of burning to a clinker was not given in the specification, and was probably not known at that time, neither was the proper proportion of clay mentioned. His success was therefore something of an accident, as was doubtless the discovery of the

engineering works. There are few suitable rocks in Europe for making this cement. It is extremely irregular in composition, and not to be compared with the very uniform beds found in inexhaustible quantities in the United States. If such natural-cement rocks as we have, had been common in England and on the Continent, it is almost certain that the artificial Portland cement would never have been discovered.

* This is really a peninsula on the south coast of England, in Dorset, near Weymouth, noted for its building-stone. The Westminster cathedral is built of this stone.

hydraulic property of the mixture itself. Aspdin began manufacturing his cement at Wakefield * in 1825.

Previous to his time a kind of natural cement had become common under the general name of "Roman cement." This was made by calcining nodules (geodes) of a clayey limestone found along the seacoast, "at a heat nearly sufficient to vitrify them," and grinding the product. (Patented by James Parker in England in 1796.)

The discovery that the hydraulic property of certain limes was due to the clay ingredient is due to Smeaton (about 1756), who had some knowledge of chemistry.† The occasion of these investigations was the building of the first Eddystone lighthouse. This, therefore, marks the beginning of all intelligent study of the subject of hydraulic cements.‡

Although Aspdin began manufacturing Portland cement in the north of England in 1825 (and continued to 1853), and it was introduced extensively on the Continent, it was not known in London till made by J. M. Maude and Son (with Aspdin's son) in 1843 under Aspdin's patents in what is now a part of London, and by J. B. White and Sons, in Kent, in 1845. §

In tests made in 1843 for the new Houses of Parliament this Portland cement was shown to be superior to the Roman cement then in common use, but engineers and architects were slow to grant the fact. Public competitive tests between the above-named firms were conducted in 1848 which further proved the superiority of the Portland cement, ‖ and after the Exhibition in 1851, at which many tests were made, its use soon became general in England.

Many failures marked the first thirty years of the Portland-cement manufacture, from an entire neglect of the chemical analysis of the ingredients. Reliance was placed solely on the empirical knowledge of workmen ignorant of chemical science, and much sophistry and deception were used to cover up their failures. It is now known that good Portland cement

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* A small city in Yorkshire near Leeds.

† A report of his investigations and conclusions was not published till 1791, in Book IV of his *Narrative of the Building, etc., of the Eddystone Lighthouse.*

‡ For a very good account of the early history of this subject see Redgrave's *Calcareous Cements*, London, 1895.

§ This was a Roman cement factory, but Mr. I. C. Johnson, their manager, after long search and experimentation, the Aspdin processes being secret and purposely mystified, discovered the secret of burning to a clinker. He also at last discovered the proper proportions. From an account by Mr. Johnson himself in *The Building News* (London), 1880.

‖ These tests consisted in building out brick beams from solid walls, and in crushing-tests of large cement prisms. As late as 1845–6 Sir Robert Peel announced in Parliament his intention of taxing the use of the clay nodules of which the Roman cement was then made, to prevent their complete exhaustion, and to retain sufficient of them for government works. Aspdin thereupon addressed him a personal note describing his artificial cement, and the proposed measure was dropped.

can be made anywhere by properly combining, burning, and grinding a mixture of carbonate of lime and a suitable clay, the only elements of commercial success being economy and scientific direction.

Since a good Portland cement, with or without sand, gravel, and broken stone, makes an artificial compound equal to almost any natural stone in hardness, strength, and durability, and since it can be moulded to any form and is much cheaper than quarried and cut stone, it is constantly finding wider and wider fields of application. This material has already worked a revolution in engineering construction nearly equal in significance to that following upon the general use of the Bessemer and open-hearth processes of making steel. The character of Portland cement also has constantly improved, until now it has reached practical perfection. Within the past ten years the improvement has been very marked, as a result of the universal system of testing now in vogue, and of the general employment of competent scientific supervision of the works, made necessary by these tests on the part of the user. Portland cement is now made on a gigantic scale in Germany, Belgium, France, and England, and its manufacture is rapidly increasing in the United States.

157. The Ingredients of Portland Cement.—All mixtures, natural or artificial, of carbonate of lime ($CaCO_3$) and clay in the proportions of from 72 to 77 per cent of the former to 20 to 25 per cent of the latter will, when calcined at the proper temperature, produce a Portland cement of fair quality. After calcining, and driving the carbonic acid (CO_2) from the carbonate of lime ($CaCO_3$), the proportions of lime (CaO) and clay (silicate of alumina (Al_2O_3, $2SiO_2$, $2H_2O$) are about 60 to 65 per cent of lime and from 25 to 30 per cent of clay, with some 5 per cent of other ingredients, such as sulphate of lime, magnesia, iron oxides, etc. "A variation in the lime ingredient of one per cent above the true amount will give a cement liable to crack on long exposure to water, and a deficiency of one per cent of lime will reduce the strength of the cement and also make the mixture liable to fuse in the kiln." * The most competent chemical supervision and continual analyses of the ingredients are therefore necessary to secure the best results.

158. Chemical Characteristics of the Ingredients.—The *carbonate of lime* should be nearly free from all other substances except clay (silica and alumina). While magnesia and iron in small amounts are not injurious, they are probably inert, and the sulphur compounds are a positive injury, above a two or three per cent limit.

159. The Clay.—"The best clays for the cement-manufacturer are those having a greasy, unctuous, feeling, quite smooth to the touch. As a rule, clays which stain the fingers should be avoided, as being either too much

* Prof. Spencer B. Newberry in the *Engineering Magazine*, June, 1894. Mr. Newberry is chemist and manager of the Sandusky, O., Portland Cement Works. This statement is probably a little too strong.

impregnated with iron compounds, or containing a large proportion of organic or other impurities. This does not hold good in the case of the carboniferous shales, some of which are rich in matters which assist in the calcination of the cement. Shales which contain much alum, selenite, or iron pyrites, and many of the shales having a high percentage of carbonate of lime, need great care in manipulation, as they are apt to fluctuate widely in composition and to lead to mistakes in the proportions of the ingredients. Some clays contain a high percentage of sandy particles, or of nearly pure silica not in combination with lime, iron, or alumina, and these clays, though useful to the brick-maker, are ill adapted for cement-making. They are generally characterized by a harsh gritty touch when tested between the finger and thumb, and it is possible to wash out a considerable percentage of sandy particles." *

160. Silica and its Compounds.—"It will be necessary, in order to understand the chemistry of cements, to treat in some detail of silica and its compounds. Silica, the oxide of the element silicon, is found very widely distributed in nature, sometimes pure, but more often in combination with other substances, as it has a great tendency to form complex salts, known as silicates. It plays the part of an acid, and combines with lime, alumina, iron, and the alkalies in a vast number of different proportions. It is found that 28 parts by weight of silicon and 32 parts by weight of oxygen are present in silicic anhydride or silica, having the chemical formula SiO_2. Clay, a hydrous silicate of alumina, may be taken as a type of the silica compounds, while quartz, flint, and chalcedony consist of almost pure silica. Porcelain clay, which contains about 47 per cent of silica, 39.2 per cent of alumina (Al_2O_3), and 13.7 per cent of water, and corresponds to the chemical formula $Al_2O_32SiO_2 + 2H_2O$, or clay proper, with a molecular weight of 258.4, may represent the silicates. There are, however, an enormous number of clays in which silica and alumina are present in very varying proportions, and which contain in addition iron, alkaline matters, lime, etc. For certain of these clays it becomes almost impossible to propound any reliable chemical formula to express their composition; and alumina, while it may combine in certain definite proportions with the silica as a base, is also capable of acting as an acid, and of combining with lime and the alkalies, especially at high temperatures, to form certain more or less unstable and little known compounds termed aluminates." *

161. "**Alumina** is the oxide of the metal aluminum which has the atomic weight of 27.2, and two parts of aluminum combine with three parts of oxygen, equal 48, to form its only known oxide, termed alumina, amounting in all to 102.4. It will not be necessary to study in detail the combinations of silica and alumina with iron and the alkalies—soda and potash—though these compounds play a very important part in cement action." *

* Redgrave.

162. Sulphur and its Compounds.—The carbonate of lime used for making Portland cement may be associated with a small percentage of gypsum or sulphate of lime (calcic sulphate $CaSO_4, 2H_2O$), the water of which is driven off in the calcining process, reducing this compound to what is commonly known as plaster of paris ($CaSO_4$). Sulphur may also be introduced in the fuel used for burning, or from the clay which sometimes contains iron pyrites. The sulphuric acid relieved from these compounds may unite with the free lime in the furnace and form an additional portion of calcic sulphate. The effect of this calcic sulphate or plaster of paris in quantities not exceeding two or three per cent is to greatly delay the time of setting (Fig. 78), but to increase slightly the final strength of the cement (Fig. 79). When present in quantities exceeding four or five per cent both these effects are lost, and it is also considered injurious in other ways, since it is comparatively soluble in water, and when present in the kiln in considerable quantity it leads to the formation of calcic sulphide, which decomposes the iron compounds in the cement, thus leading to disintegration. The German standard rules allow a proportion of calcic sulphate not to exceed two per cent, but an effort has recently been made to have this limit raised to three per cent.

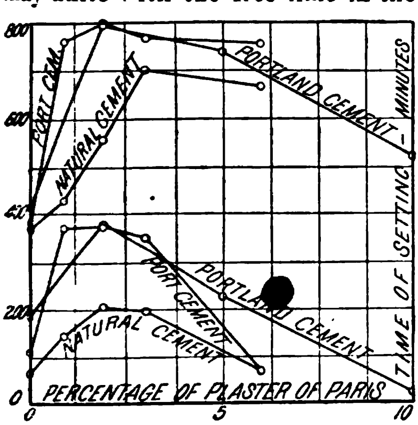

FIG. 78.—Effect of Plaster of Paris on Time of Setting of Cement. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2938.)

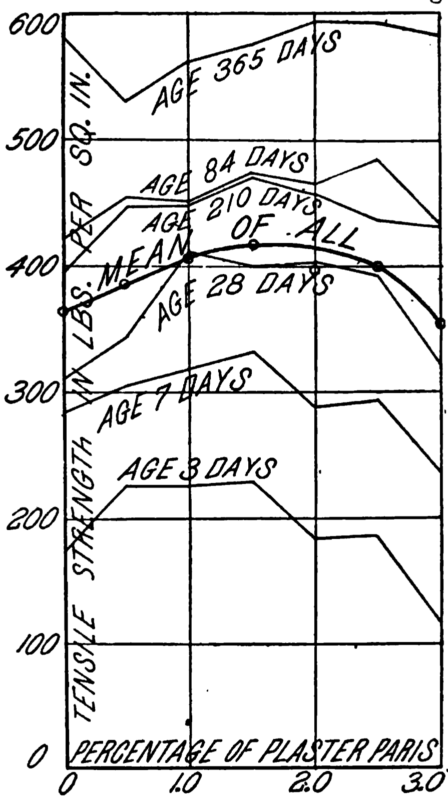

FIG. 79.—Showing the Effect of Plaster of Paris on the Strength of Portland-cement Mortar, 1 C. : 3 S. (Tetmajer, vol. VII. p 39.)

163. The Chemical Reactions Produced in Calcining.—It has been commonly agreed by chemists that the following combinations are effected by calcining an intimate mixture of lime and clay to the point of incipient fusion:

	Proportions by Weight.	
Silicate of lime (SiO_2, $3CaO$)	Silica	23
	Lime	43
Aluminate of lime (Al_2O_3, $3CaO$)	Alumina	17
	Lime	28
Double silicate of lime and alumina ($SiO_2(Al_2O_3+CaO)_2$)	Silica	15
	Alumina	51
	Lime	28

Magnesia probably remains inert, and does not combine with alumina and silica. It is harmless if not forming over five or six per cent of the whole.* Oxide of iron is also a useless ingredient.

M. Le Chatelier has been able to identify the aluminate of lime by a microscope, with polarized light, in both cement clinker and in an artificial synthetic compound. He also thought he determined two other substances in this manner, they being a silicate of lime ($2CaOSiO_2$) found crystallized

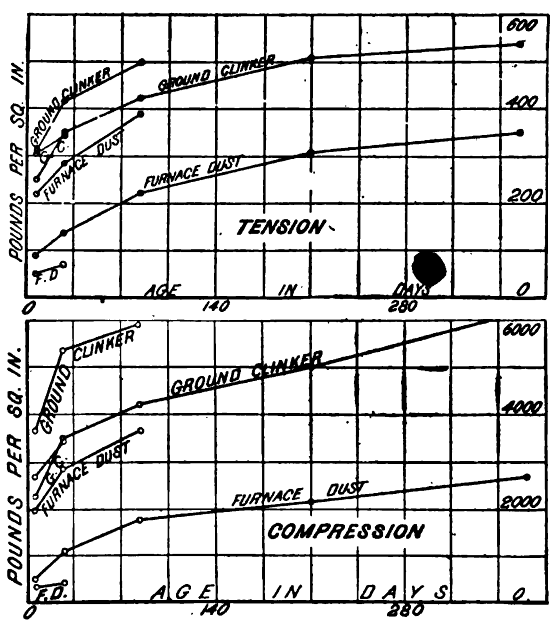

FIG. 80.—Showing the Inferior Character of the Furnace-dust compared with the Ground Clinker, when used in Mortar, 1 C.: 3 S. (Tetmajer, vol. VII. p. 12.)

in a matrix of an alumino-ferrite of lime, with a formula $2(AlFe)_2O_3 3CaO$. These chemical reactions in the furnace are, however, not yet known with certainty.

Since there is no further mixing of the lime and clay ingredients in the furnace in the calcining action, it is absolutely necessary, in order to secure

* The German Cement Manufacturers' Association has allowed five per cent since 1893.

perfect results, to have the lime and the clay perfectly and uniformly mixed before going into the furnace; that is to say, each particle of lime should have adjacent to it its particle of clay with which to unite when the proper temperature has been attained. Since it is, of course, impossible to intermix these materials to this degree of perfection, there must of necessity result from the burning more or less inert or uncombined clay and lime without cementing qualities, which inert matter forms a large part of the furnace-dust. (See Fig. 80.) If the ingredients were actually fused or melted into a liquid mass, and the chemical action were to take place after the ingredients were in the liquid form, a much more perfect union of the elements would of course be effected. In the formation of the clinker which is ground into Portland cement, however, the ingredients are not fused, since fusion would be fatal, and hence the elements of the mixture are incapable of uniting except they be in immediate juxtaposition. The further improvement of Portland cement evidently lies in the direction of more perfect and more uniform mixture of the raw materials in a finely divided state before they are burned. From experimental tests which have been made in this direction, it would seem that the strength of Portland cement might be made at least twice what it is now, by more perfectly satisfying this requirement.

164. The Chemical and Physical Changes involved in Setting and Hardening.—By the *setting* of cement is meant its initial change from a soft or plastic mortar to a friable solid. This change is usually effected with great suddenness, after it begins, as shown by the curves in Fig. 333, and it has been shown to be always accompanied by the evolution of heat. After the cement has become thoroughly set it still is very weak, and is readily pulverized in the fingers. If left undisturbed, however, it increases in hardness and strength, sometimes for several months, but generally for many years. There is no relation between the time elapsing after wetting before setting takes place, and the period of time required to attain to nearly its ultimate strength. *The setting of cement is thought to be effected by the crystallizing out of the silicate and the aluminate of lime, which are soluble in water in their anhydrous form.* After dissolving in the water they pass to the hydrated state in which they are insoluble, and hence are precipitated in a crystalline form, with a development of heat. This process is greatly hastened at higher temperatures.

The *hardening* of cement is due to a continued crystallization of salts from solution, and to further chemical and physical changes which develop slowly, but which continue for long periods of time. M. Fremy regards the aluminate of lime as the chief source of the hardening property, and he also thinks the silica and the alumina of the clay are separated by calcining and take on allotropic forms, ready to unite into new compounds with the quicklime when water is added. There are so many kinds of combinations of various substances which will serve to produce the final characteristics of

hardened Portland cement, that there must be many different chemical compounds which, after calcining, will harden on the addition of water. The problem is so complicated that it has as yet defied a complete chemical analysis.

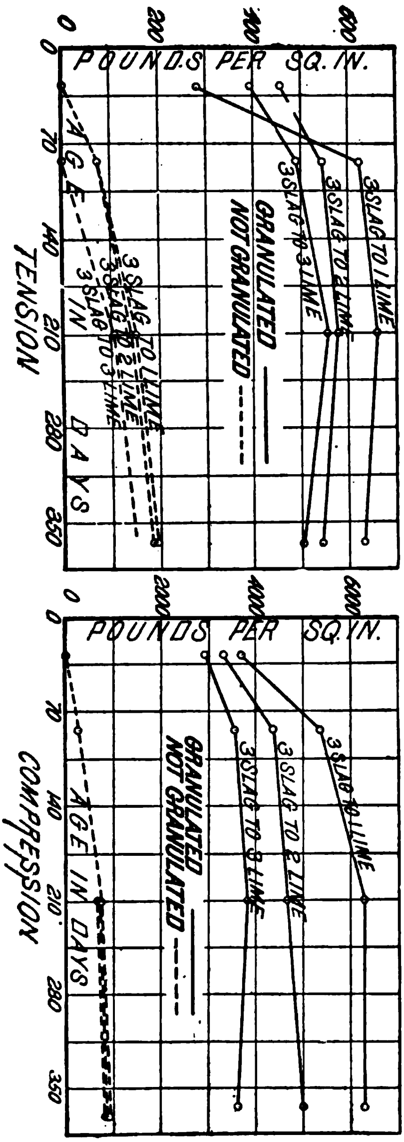

Fig. 81.—Showing the Beneficial Effects of "Granulating" Slag-cement by running it into Water in a Melted State when made into Mortar 1 C. : 3 S. (Tetmajer, vol. VII. p. 95.)

165. Slag-cements.—Many varieties of iron blast-furnace slags will make an excellent cement when ground with hydrated or slacked lime, without further calcining. The slag is "granulated" by running it from the blast-furnace into water, where it forms into a brittle, porous, pumice-like mass resembling caked sand, and in this condition it is called "slag-sand." It is now easily crushed into powder, but retains the water in its meshes so that it is very difficult to dry it. Sometimes this "slag-sand" is calcined at a low heat simply to dry it.

This method of suddenly cooling the melted vitreous slag in water has no effect upon it further than to simply reduce it to the porous, friable condition, since when allowed to cool in the ordinary way, into a solid mass, and then crushed to powder and lime added, it has no hydraulic properties (see Fig. 81). It seems probable, therefore, that the sudden cooling leaves the chemical compounds in a more unstable condition, so that when powdered and intimately mixed with hydrated lime, and water added, they are ready to enter into new chemical combinations with the lime. To three parts by weight of the dry "slag-sand" is added one part of hydrated lime (CaH_2O_2), and these are thoroughly ground together and intermixed by suitable mechanical appliances. This cement does not deteriorate appreciably by lapse of time.

The hardening properties of slag-cement, with its small proportion of lime, and its being an artificial mixture of free lime and slag without subsequent calcining, have greatly disturbed the previously accepted theories of the chemical transformations in the furnace and after melting, in the case of Portland cements, and it seems that now the whole matter will have to await a further progress of chemical knowledge in this direction.

Slag-cements make a more unctuous mortar and are much liked by architects for laying brick walls and piers, and for making floors, sidewalks, etc. It is slow setting and does not stain the masonry in outside walls. It is often mixed with additional amounts of lime-putty to further delay the time of setting, or to cheapen the mortar, or to make it work smoother under the trowel, or perhaps for all these reasons combined.

166. Sources of the Raw Materials Used in Making Portland Cement.*— "Portland cement is made from carbonate of lime and clay. These materials may be naturally mixed, as in the case of argillaceous limestones, or entirely separate. In all cases, however, it is necessary to bring the material to correct composition by artificial additions and thorough mixing. In England chalk is the form of carbonate of lime employed. In Germany the chief material is marl (mergel), by which is understood a more or less hard limestone rock containing clay. In some German factories a pure soft marl (weisenkalk), or fresh-water chalk, is used, consisting chiefly of carbonate of lime and similar to the marl deposits of this country.

"In the United States the materials used are very similar to those of Germany. Most of our clay limestones are highly magnesian, and therefore unsuitable for Portland cement, though they are used on an immense scale for natural-rock cements. At certain localities, however, as in Lehigh County, Pa., at Phillipsburg, N. J., and in the far West, limestones containing sufficient clay and nearly free from magnesia are abundantly found, and in the above localities and from this material most of our Portland cement is made. In the Lehigh County region, the chief seat of the American Portland-cement industry, the different strata of rock are carefully selected and mixed in such proportions as to give a material of the right composition.

"In central New York and at a few points in Ohio and Indiana large deposits of pure white marl are found. This is generally called 'shell-marl,' and was supposed to result from the disintegration of fresh-water shells. In the opinion of the writer, however, these marl-beds are generally pulverulent deposits from calcareous springs, and are not formed from shells to any great extent. At the localities above mentioned this material, artificially mixed with clay, is largely used for the manufacture of Portland cement. Owing to the soft, fine-grained character of the marl, the mixing can be much more cheaply done than in the case of limestone, though this

* This and the following article are taken from the paper on Portland Cement by Prof. Spencer B. Newberry, in the U. S. Geol. Surv. Report for 1894, Part IV, p. 581.

advantage is largely compensated for by the necessity of drying out the 40 to 50 per cent of water which the marl generally contains. It must be remembered also that in the argillaceous limestones the ingredients are already uniformly mixed in nearly the proper proportions, while with the pure lime and clay this mixing must be wholly effected by artificial means. The leaving of any free lime in the final product, from imperfect mixing, has often led to the disintegration of the mortar by sea-water, and by fresh water containing carbonic acid in solution.

"As already stated, most American Portland cement is made from argillaceous limestone, as shown by the following table.

NUMBER OF AMERICAN CEMENT FACTORIES USING LIMESTONE COMPARED WITH THE USERS OF MARL (1894).

Factories Using	Number.	Quantity.
		Barrels.
Limestone............. ...	17	611,829
Marl........................	7	186,928
Total..................	24	798,757

"The first group includes 6 factories in the Lehigh County region in Pennsylvania, producing over 400,000 barrels; 1 at Phillipsburg, N. J.; and 10 at other points. The second group, using marl, includes 4 factories in New York, 2 in Ohio, and 1 in Indiana."

167. Processes Used in Pulverizing and Mixing the Raw Materials.— There are, in general, three processes employed in preparing this intimate mixture of the raw materials, which may be designated "The Wet Process," "The Semi-wet Process," and "The Dry Process."

1. *The Wet Process* was originally employed in England and in France, and was used for the admixture of crushed chalk and clay. These were pulverized and mixed in "wash-mills" with such an excess of water as to form a thin liquid. This was stirred by such an arrangement as that shown in Fig. 82, the escape being at the top over a lip or weir. The coarsest particles settled in this wash-mill, and such granulated matter as escaped in the liquid was intercepted on its way to the "backs," which were open tanks some four feet deep, with earth or gravel bottoms. The mixture was now allowed to settle for some days, when the clear water was siphoned off and the "slurry" left to dry in the open air until it could be handled with a shovel. It was then wheeled upon drying-floors and dried by artificial heat into irregular clods or masses, when it was sent to the furnace. Even in summer this process required many weeks' time for its completion, and in the first settlement the chalk and clay ingredients would sometimes have a different specific gravity, and hence they would not settle simultaneously. This would give an uneven mixture, which would be so far fatal, since it

could not be corrected. This process is going out of use even for such material as is suited for this method of treatment.

2. *The Semi-wet Process* consists in mixing the ingredients in the state of a soft paste. This may be done either by grinding them together in this condition or by means of "edge-runners." These consist of heavy cast-iron cylinders of short length, mounted on a horizontal axle which is made to

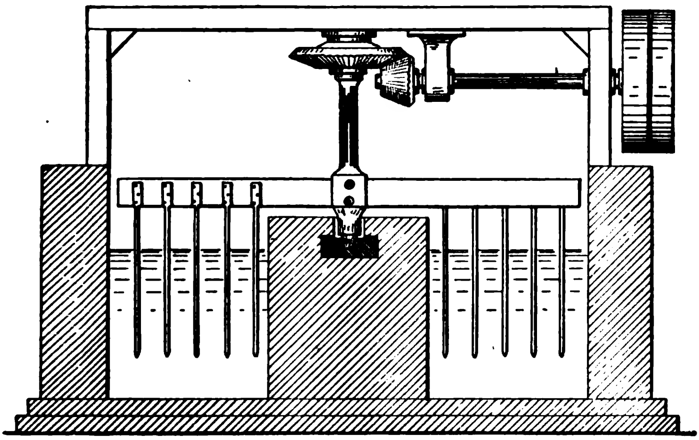

FIG. 82.—Wash-mill used in the Wet Process of making Portland Cement.

swing about a vertical axis, thus causing the heavy cylinder to roll about on a bed-plate. Sometimes the roller-axle maintains a fixed position and the plate revolves on which it rests. This is an efficient pulverizer and mixer when the ingredients are comparatively soft. It does not produce as uniform a pulverization, however, as a grinding-mill. Both these processes are used in America.*

3. *The Dry Process* is used in Germany, and in Pennsylvania and New Jersey in this country, where the materials consist of argillaceous limestone having nearly the proper composition for making Portland cement. The rock is first crushed and then ground. The final mixture of limestone and clay-shale is made before the material is ground, so that the process of grinding effects a very thorough mixing. The ingredients must be reduced to an impalpable powder in order to make possible that thorough mixing necessary to enable each molecule of lime to associate itself with its molecule of clay in the calcining process, so as to produce the true chemical combinations in the clinker. If the limestone used has primarily nearly the composition required, which is sometimes the case in America, then it is evident that any want of perfection in the first grinding and mixing of the raw materials is not so injurious, since the native mixture is not only

* At Bellefontaine, O., the "edge-runners" are used, and at Sandusky, O., the grinding-mill, the material in both cases being a soft marl and clay.

nearly correct as to proportions, but so far as it goes the admixture is practically perfect. When pure carbonate of lime is used (as in the case of soft marl) with clay, there is no primary mixture of the lime and clay at all, and hence the necessity of a much more elaborate artificial mixing process than when these ingredients are found intimately associated in a natural rock and to nearly the correct proportions. On the other hand, the soft marl and clay (of Ohio, for instance) are much more easily worked than the hard limestone and clay shales (of Pennsylvania). In order to enable the hard materials to compete successfully with the soft, it is necessary that the limestone should contain primarily nearly the proper proportion of clay.

After the dry grinding and mixing of the raw materials, the dust is wet sufficiently and moulded into bricks (thus obtaining a further mixing), and then dried and burned. The raw powder cannot be calcined in the ordinary furnaces without first compacting it in aggregate forms to allow of a draft of air through them. In the tubular rotating furnaces it is calcined as a dry powder, and this is one of the great advantages of that process.

168. Processes Used in Burning Portland Cement.—"There are three distinct forms of kiln used in burning Portland cement in America. These are (1) intermittent or dome kiln, (2) continuous kiln, of the Dietzsch or Shöfer type, (3) rotary furnace. In the old-fashioned intermittent kiln the bricks of cement mixture are charged into the kiln with coke in alternate layers, and the whole allowed to burn out and cool down before emptying. The Dietzsch or Shöfer continuous kiln is continuously charged with bricks of cement mixture and soft coal, and the burned clinker periodically withdrawn at the bottom. It presents the great advantage of cheaper fuel and economy of labor, and burns the dry powdered material. The rotary furnace consists of a rotary cylinder heated by a blast of air and gaseous fuel, the material being continuously run in at one end, and issuing as burned clinker at the other. This process was patented by Mr. Frederick Ransome, in England, in 1885, and has been subsequently modified and improved by others. Many difficulties have been met with in carrying out this plan, but it is now successfully operated at a number of works in this country. It would seem to be the most rational method of carrying on the burning of cement, since it effects an enormous saving in time and labor, and allows the temperature to be regulated far more exactly than is possible in the older processes. Crude or fuel oil is used as a source of heat at all points in America where this kiln is employed, though producer-gas with or without regenerative furnaces might be employed.*

"In the United States most of the Portland cement produced is burned in the old-fashioned intermittent kilns. The Dietzsch kiln is used at Harper and Middle Branch, Ohio. The Shöfer kiln is to be used at new works now

* This process requires a much greater fuel expense than the kilns and seems to be used only where the raw material will not adhere sufficiently by wetting to form briquettes which can be burned in kilns.—J. B. J.

beginning operations at Glens Falls, N. Y. The rotary furnace is in operation at Colton, Cal., Phillipsburg, N. J., Coplay, Pa., and Sandusky, Ohio. The following table shows the number of barrels of cement made during 1894 and 1895 in vertical kilns (continuous and intermittent) and the rotary furnace.

AMOUNT OF PORTLAND CEMENT MADE IN KILNS OF VARIOUS KINDS.

	1893. Barrels.	1894. Barrels.
Rotary furnace	149,000	242,176
Vertical kilns (continuous and intermittent)	441 653	556,581
Total	590,653	798,757

It thus appears that the output of rotary furnaces has increased much more rapidly than that of vertical kilns. The recent rapid advance in the price of crude oil is a great obstacle to the use of the rotary furnace. Attempts are being made to substitute producer-gas for crude oil in burning cement. There is no reason why this should not be successfully done, and the change will greatly reduce the cost of burning cement at all points where the rotary process is used."

169. Grinding the Clinker. — "For grinding the finished product the Griffin steel mill is used at the larger factories. Some of the older works still use buhrstones. The Griffin mill* consists of a steel ring, against the inside surface of which a heavy steel roll revolving on a vertical shaft presses *by centrifugal force.* Fig. 83. The mill is provided with screens which allow powder of the requisite fineness to pass through, while the coarser particles drop back into the mill. This mill is an American invention, and is rapidly finding its way into the leading cement-works of Germany."

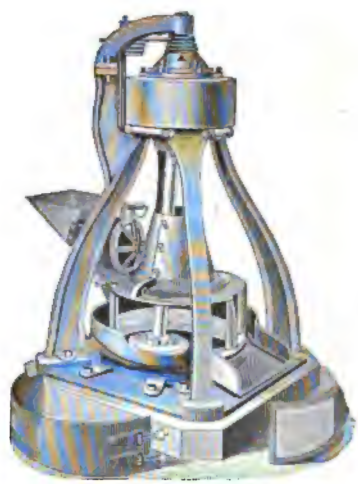

FIG. 83.—Perspective View of the Griffin Mill.

* Made by the Bradley Pulverizer Co., Boston, Mass. It is used for all kinds of pulverizing where buhrstones and stamp-mills have hitherto been employed. It works in either wet or dry material, and is an extremely ingenious and successful grinding machine.—J. B. J.

CHAPTER XII.

THE MANUFACTURE OF VITRIFIED PAVING-BRICK.

By H. A. Wheeler, E.M.*

170. Definition.—As there is a lack of harmony in the use of the term vitrified brick, it is necessary to define what is meant by *vitrified.* There is a popular idea that a vitrified brick must be *glassy,* in accordance with the etymology of the word; whereas a truly glassy brick is impracticable to make—at least to a reasonably large percentage; and unless annealed with very much more care than is now given to paving-brick, such a brick would be too brittle for paving purposes, besides being badly misshapen. It is true that samples of excellent paving-brick frequently exhibit to an eminent degree a glassy or vitreous surface; but these vitreous faces are due to air-checks (caused by the hot brick being struck by cold air), and if the brick is broken along an unchecked or solid face it will not exhibit a glassy surface: it will there present a very close, dense, homogeneous, stone-like fracture, and this fracture is what is recognized and accepted as characteristic of a vitrified brick. There is a total absence of the individual particles of the clay in such a fracture the presence of which characterizes building and fire-brick. Furthermore, such a vitrified brick has a hardness of 6.5 to 7, on Moh's scale of hardness, or is about as hard as quartz (the hardest mineral in granite), and it readily scratches glass or the hardest steel. While this typical vitrified fracture is easily recognized by the experienced eye, there is no sharp line of demarcation between it and the glassy fracture on the one side (when the brick is *overburned*), and a hard but unvitrified brick on the other hand (when *underburned*); for clay *gradually passes* through a transition, when highly heated, from (1) an eminently porous, strong, and rather hard condition just previous to the vitrifying-point; (2) to a very much harder, tougher, slightly porous condition when vitrified; and finally (3) to a very dense, glassy, non-porous condition when completely vitrified, in which latter condition it is very apt to be decidedly brittle. These three stages of burning can usually be found in every kiln of paving-brick, with all intermediate transitions from one extreme to the other,

* Formerly Assistant Geologist Missouri State Geological Survey, in charge of investigations made on clays, and now (1896) manufacturing paving-brick in St. Louis, Mo.

though from 60% to 90% usually come within the second or properly vitrified stage.

171. Clays employed for Paving-brick.—Three radically different classes of clays are employed in the manufacture of paving-brick, viz.:

I. Surface Clays;
II. Inferior Fire-clays;
III. Shales.

Surface Clays.

By surface clays are meant those soft, unconsolidated clays at or near the surface which have been deposited during or since the glacial period, or that have resulted from the atmospheric decay of the underlying rocks. This class of clays was more frequently used in the earlier development of the paving-brick industry, but they have been almost completely given up on account of the great difficulty in successfully vitrifying a large percentage of the brick; for, as a rule, they are apt to be so very siliceous (or have from 60% to 80% silica), or else so very calcareous (or have from 10% to 25% lime), that there is usually a very narrow range of temperature at which they can be vitrified. Hence the brick are apt to be either too soft (underburned), or else overburned and badly misshapen, so that these clays have been generally abandoned for the safer-burning shales and fire-clays.

Inferior Fire-clays.

The inferior or impure fire-clays, which are frequently known in the trade as "bastard fire-clay" or "pipe-clay," have been quite largely used in the past, and are still employed to some extent in the manufacture of paving-brick. They are a class of fire-clays that contain sufficient fluxing impurities to enable them to be slightly vitrified, and the more impure the fire-clay the more successfully it can be used for this purpose. When the physical properties of the fire-clay are suitable, these impure fire-clays make an excellent quality of fire-brick, though they always show rather high absorption, or from 2% to 5% of water after soaking 24 hours in water. They never show a glassy fracture, and are rarely misshapen or kiln-marked, as in the other two classes of clays; but they are very much more apt to be soft, and therefore short-lived, from underburning, as it requires a very high heat to vitrify them. When properly vitrified, they make a very satisfactory paving-brick, on account of their toughness, and some of the oldest paving-brick in the country were made from this class of clays, notwithstanding that they exhibit a very high absorption of water.

Shales.

The shales, or those hard, consolidated, laminated, rock-like clays that are also popularly called "soapstone" and "soft slate," are now almost exclusively used in the manufacture of paving-brick. They occur in very

much larger and thicker bodies than either the surface clays or fire-clays, often outcropping as low hills, and they can usually be cheaply worked by steam-shovels in open pits. While they are usually non-plastic as they occur in the bank, they can be easily ground to powder, when they readily work up into a plastic mass with water. The shales are usually very high in fluxing impurities, and this is the reason why they are so favorably adapted for paving-brick, as this enables them to be readily vitrified. The average composition of the shales that have proved eminently satisfactory for this purpose is as follows:

Silica (SiO_2)	56 per cent.	
Alumina (Al_2O_3)	22 "	
Ignition loss (chemically combined water)	7 "	
Moisture (H_2O)	2 "	
Total non-fluxing constituents		87 per cent.
Sesquioxide of iron (Fe_2O_3)	7 per cent.	
Lime (CaO)	1 "	
Magnesia (MgO)	1 "	
Alkalies (K_2ONa_2O)	4 "	
Total fluxing constituents		13 per cent.
Grand total		100 per cent.

While the best shales range quite closely around the preceding analysis, quite a range in the fluxing constituents is permissible, as the chemical analysis is always *very secondary* in the consideration of clays. All clays, for any purpose whatever, depend primarily on their physical properties, and if these are not favorable the chemical composition is of no importance. A very elaborate discussion of the chemical composition and the influence of the impurities of clays is given by the writer in Part I. of the "Report on the Missouri Clays," to which the reader is referred for details which it is impossible to discuss in this brief chapter.*

172. Physical Properties of Clays.—The physical properties of clay, on which depend its manufacture and uses, consist of the following factors:

I. Plasticity;
II. Shrinkage in drying and burning;
III. Speed in drying, burning, and cooling;
IV. Point of incipient, complete, and viscous vitrification;
V. Density before and after burning;
VI. Colors of burned ware;
VII. Strength of burned ware.

Plasticity.

Plasticity is the most important quality of any clay, as its ability to be moulded depends upon this property. When mixed with the proper

* To be obtained from the State Geologist, Jefferson City, Mo.

amount of water it is called *fat* when it is very plastic, and the more plastic the clay the stronger the brick will be. In making paving-brick, excessive plasticity is found to increase the defect of laminations, which is a great source of weakness if excessively developed. To counteract this trouble the clay is either mixed with a less plastic one, or with sand, "grog," or other lean materials which reduce the plasticity.

Shrinkage.

The shrinkage is a very important factor in determining the size of moulds and dies to produce a given-sized brick after burning. The drying shrinkage is the reduction of volume which takes place when the soft mud brick becomes dry from the elimination of the water used in moulding, which amounts to 3 to 7 per cent. A second shrinkage occurs when the dried brick is burned, which is greater the harder the brick is burned, until thoroughly vitrified, when it ceases to shrink. The fire shrinkage varies from 4 to 8 per cent, and the *total* shrinkage ranges from 7 to 15 per cent.

Speed of Drying, etc.

The speed of drying, burning, and cooling are extremely important factors to the manufacturer in determining the size of his plant, besides being of great importance in affecting the strength of the brick. Some clays can be rapidly dried, burned, and cooled without having their strength seriously impaired, while others are very much weakened, if not actually cracked or ruptured, unless this is carried on very slowly. As a broad rule, the more plastic a clay the more slowly it must be dried, burned, and cooled, while the coarser and leaner clays can be treated much more rapidly without detriment. This is a factor that is keenly appreciated by the manufacturer, but is rarely understood or appreciated by the engineer, yet it affects the strength of the brick more than any one factor. It does not follow that a clay that requires to be slowly dried must necessarily be slowly heated and cooled, or *vice versa;* for there is an individuality about clays that requires a separate determination of each of these factors as to their amount and influence.

Vitrification.

The stages of (1) incipient, (2) complete, and (3) viscous vitrification are extremely important, as paving-brick should be raised to at least the stage of incipient vitrification to secure the requisite density, hardness, low absorption, and toughness; while it must not be raised to the point of viscous vitrification, as it then loses its shape. In the shales suitable for paving-brick the first stage is reached at from 1500° to 1800° F., and the second at 1800° to 2200° F., or at a very bright cherry-red.

Density.

The denser the clay the denser the brick made therefrom will be, and the higher the density the more durable the brick. The specific gravity

of shales usually range from 2.10 to 2.60, and the specific gravity of the brick will be about the same. The impure fire-clay brick are generally somewhat lighter, or vary from 1.95 to 2.30, but the brick have a specific gravity somewhat lower than that of the original fire-clay.

Color.

The color of paving-brick is of great local importance in estimating the degree to which it has been burned, and the care with which it has been handled. If the shale is high in iron (which is usually the case) the resultant brick varies from red to very dark brown in color, while if the clay is low in iron and high in lime it is light in color. Furthermore, the skill of the burner is able to largely influence the color by the manipulation of his fires, so that general rules for determining the quality of paving-brick by color only are dangerous, though for specific cases and a given burner they are of very great aid in quickly arriving at the quality of the brick.

THE MANUFACTURE OF PAVING-BRICK.

173. Preparing the Clays.—The surface clays are usually obtained by either the pick and shovel, plough and scraper or clay gatherer, or the steam-shovel and cars, according to the size of the yard and local conditions. The fire-clays, as they usually occur underground, are mined by the room-and-pillar system, like coal, which is very much more expensive. The shales are sometimes worked by the room-and-pillar system, where they occur underground, but in most cases they are worked in open pits, by blasting, or else worked direct from the bank into the cars by powerful steam-shovels.

The clays are sometimes pulverized by toothed rolls, and occasionally by centrifugal disintegrators, but in most cases a revolving dry-pan with a perforated grate bottom is employed especially for shales and fire-clays.

The crushed clay is usually screened in either revolving trommels, or fixed or shaking riddles, with 4 to 16 meshes to the linear inch. The degree of fineness of the screen is a very important matter, as the finer the clay the more plastic it is, and hence the stronger the brick. In some cases, however, excessive fineness causes checking and cracking in drying or burning, and aggravates the trouble from laminations, so that the fineness of the screen should be determined for each specific clay. Sometimes the clay is not screened any further than is accomplished by the screen-plates of the dry-pan, which are usually $\frac{1}{8}$ to $\frac{1}{4}$ inch in width.

The screened clay is next mixed with water to a more or less plastic mass in a pug-mill. The pug-mill consists of a trough containing a revolving shaft that is armed with blades set at an angle. It should revolve at such a speed, or the blades should be set at such low pitch, or the length should be sufficiently great, or the amount of clay to be pugged should be so restricted, as to secure a *thorough, uniform mixture* of the clay and water; but frequently the pug-mills are too short to accomplish this, or they

are overcrowded, or speeded too high, or the clay is run through too quickly by the blades being given an excessive pitch, and consequently the clay comes out with variable amounts of water. This causes checking and cracking in the drying, and sometimes in the burning, with marked variations in the strength of the brick, besides causing the bar of clay to rag as it leaves the brick machine. The more thoroughly a clay is pugged, the more plastic it is rendered, and the more uniform and reliable will be the quality of the brick, and this department could be remodelled to decided advantage in most paving-brick plants.

174. Moulding.—Three processes are employed for moulding paving brick, to wit:

The Soft-mud Process;
The Stiff-mud Process;
The Semi-dry Process.

In the Soft-mud Process the clay is mixed with sufficient water to make a very soft, extremely plastic mud, which is moulded by hand or soft-mud machines into imperfectly formed brick; these are allowed to partially dry, to a firm, stiff condition, and are then repressed into perfectly formed brick. This process makes an excellent quality of brick, but it necessitates a second handling of the brick during the drying stage. Outside of a few small yards, it has been quite generally given up in the paving-brick trade, on account of the expense of the extra handling and breakage, besides considerable risk of injuring the strength of the brick, if they are allowed to get too dry.

In the Stiff-mud Process the clay is pugged with sufficient water to make a stiff, plastic mud, which is forced through a die by a continuous-working auger or intermittent plunger, as a bar of clay, which is then cut by wires into suitable lengths. This is the process that is almost universally employed in the manufacture of vitrified brick, as the mud is stiff enough to be made into perfectly shaped brick, which can be loaded on to cars without risk of being marked or injured in handling.

In extruding the bar of clay from the brick machine, two types of dies are employed: in one the bar of clay is approximately 3″ × 4″ in section, which is cut into 9″ lengths, and is known as the "end-cut system"; while in the other the die is approximately 4″ × 9″ in section, and the bar is cut into 3″ lengths, which is known as the "side-cut system." There is considerable difference of opinion as to the relative merits of these two methods of moulding, which too frequently is founded on very diversified facts. If the clay is lean or sandy, the side-cut brick is apt to be of better quality than the end-cut; while if the clay is very fine and eminently plastic, the end-cut system gives fewer laminations and a superior quality to the side-cut.

In the Semi-dry Process the clay is mixed with just enough water to dampen it, so that it adheres slightly when firmly pressed. The brick are

moulded by feeding the damp clay into a mould-box, in which it is subjected to a very heavy pressure by a reciprocating plunger. This process has been used to a very limited extent for paving-brick, as the brick are not as tough as when made by the mud process, while it is much more difficult to burn a large percentage of No. 1 grade. As there is considerable difficulty in feeding the mould-boxes with damp clay, the moulds are frequently only imperfectly filled, which prevents the brick from receiving the heavy pressure necessary to bond it, besides causing imperfect faces.

Repressing.—Recently there has been a heavy demand for repressing the brick made by the stiff-mud process immediately after it leaves the brick machine. In the repressing process the brick is exposed to a moderate vertical pressure in a metal mould-box, while still in a plastic condition, which thoroughly fills out the edges and angles, and rounds them if desired. This results in a brick of uniform size and perfect shape, so that the appearance of the brick is greatly improved ; but as the pressure is moderate, it is doubtful if the quality of the brick is enhanced by this extra operation. Where there have been opportunities for testing the relative merits of the same clay in repressed and unrepressed brick, the facts indicate that the strength of the brick is endangered by breaking the structure formed in the slow-acting brick-machine by subjecting it to such radically different forces as occur in a vertical-acting, quickly applied repress. Thus some Purington *unrepressed* brick have been exposed for two years on Lasalle Street, Chicago, to the heaviest kind of metropolitan traffic, which it has very successfully withstood; while *repressed* brick from the same plant has not stood so well on other Chicago streets with much less traffic. This is a matter that needs further investigation and more facts, and the above is the most important evidence known to the writer that bears directly on this question. *

175. Drying and Burning.—*Drying.*—The moulded brick are hacked on cars, in open checkerwork, direct from the brick-machine, which are run in drying-tunnels, where they are exposed for 24 to 60 hours to light open fires, or to a heated blast, or to the radiation of an extensive series of steam-pipes, in order to expel the water used in moulding. Some clays can be safely dried in 18 to 30 hours without checking or cracking, while others have their strength seriously impaired unless the drying takes from 48 to 72 hours. Usually the finer and more plastic the clay the greater the time required, while the coarser and leaner the clay the more rapidly it can be dried.

Burning.—Three classes of kilns are employed in burning: the up-draught, the down-draught, and the continuous. The up-draught kiln, which is the type usually employed in burning building-brick, has a series of parallel fires at the bottom of the kiln, from which the heat rises through the brick, and escapes at the top of the kiln. The brick that are directly exposed to the fire at the bottom receive too much heat, while the brick at

* Recent (1896) rattler tests by Prof Orton, on bricks made from the same clay, on different machines, and burned together in the same kiln, indicate clearly that repressing an end-cut brick benefits it, while repressing a side-cut brick injures it.

the top of the kiln do not receive sufficient heat to vitrify them. There is, consequently, a goodly percentage of overburned, misshapen brick at the bottom of the kiln, and a heavy percentage of soft, unburned brick at the top, the central portion being the only part that receives the proper degree of heat. As the percentage of No. 1 brick, or those suitable for paving, ranges from 35 to 65 per cent, according to the skill of the burner, the up-draught type of kiln is seldom employed for paving-brick. In the down-draught type of kiln the heat rises to the top or crown of the kiln from a series of outside fires, and then passes down through the brick to flues at the bottom of the kiln, and then escapes to one or more stacks. The brick are protected from excessive heat, and the heat is more thoroughly and completely distributed through the brick than in the up-draught type. The percentage of first-class brick is therefore much greater, as with intelligent handling from 60 to 90 per cent of No. 1 pavers can be obtained.

The down-draught kilns were formerly of the round or beehive type, which hold from 25,000 to 75,000 brick; but in recent practice the long rectangular design is preferred, which hold from 100,000 to 300,000 brick, and most of the paving-brick are burned to-day in kilns of this design.

In the continuous type of kiln the coal is fed directly in among the brick, which are piled in a long tunnel, and the heat is drawn through them by a high stack at the opposite end. This results in a great economy of fuel over both the up-draught and down-draught types of kiln; but the shrinkage and the difficulty of securing uniformity in burning is so great that they only yield from 40 to 70 per cent of No. 1 pavers, and they are not generally used.

The practice of glazing paving-brick with salt, similar to sewer-pipe, was formerly employed to a considerable extent, as it gave the brick a dark color, which was supposed to indicate hardness, besides rendering defects less conspicuous. As the glaze is superficial, it adds nothing to the durability of the brick, while it greatly increases the difficulty of sorting by color, and enables soft brick to be overlooked unless very thoroughly inspected. The practice is to be strongly deprecated, and it is dying out.

176. Annealing.—After the brick have been burned, the kiln should be tightly closed to shut off the access of cold air, and the longer the time given the brick to cool and anneal the tougher the brick will be. Bricks made from the best clays can be ruined by cooling off the kiln too rapidly. If this does not result in checking the brick it will at least make them brittle. The conductivity of clay for heat is so feeble that unless the brick are very slowly cooled internal stresses are produced—very much as in rapidly cooled steel or glass—which interfere with the toughness of the brick. This annealing or toughening by slow cooling is not appreciated by engineers, though well understood by the brickmakers. They claim they cannot afford to take the time in cooling off the kilns that they would like to, where the price is the criterion that will determine the successful bidder, while quality is made subordinate. The kilns are the most expensive portion

in a paving-brick plant, and delay in emptying them by slow cooling adds considerably to the expense of manufacture; so that unless the brickmakers are paid accordingly, they cannot afford to anneal with the care that is demanded for the best quality of brick. The usual practice in brickyards is to "turn" or fill a kiln once a month, which allows from six to nine days for cooling off. If the kiln capacity of a yard could be increased 25 per cent so as to give the kilns twice the time to cool off, it would result in a very much tougher, more uniform, and reliable brick; but it would necessitate a price commensurate with this increased outlay of capital, as there would be no increase in the quantity of brick produced.

Where the very best quality of paving-brick is required, a matter of $1.00 or $2.00 increase in the cost of the brick to insure thorough annealing would prove to be very great economy, and a very judicious investment, in greatly increasing the durability of the pavement.

177. Sorting.—In emptying the kiln there are usually three grades of brick made in the vitrified trade. In the down-draught type of kiln, one or two top courses are liable to be air-checked and more or less brittle if the kiln is either improperly designed or improperly handled in burning, while the top layer is always covered with soot and ashes that mars and stains the surface of the brick. As these brick get the highest heat they are usually the hardest, and while not generally tough enough for paving purposes, they are very desirable brick for sewers, foundations, and sidewalks, especially as they are free from kiln-marks, and are seldom misshapen. The first two or three layers are therefore usually set aside and sold as sewer and sidewalk brick.

The bottom portion of the kiln, or the lower two to ten courses, do not usually receive sufficient heat to be properly vitrified, and are known as No. 2 or building brick, as they are well adapted for foundations or for backing-stock brick.

The intermediate or central portion of the kiln are No. 1, or strictly first-class, paving-brick, which are distinguished by the fracture, toughness, hardness, and the color from the other two grades of brick. They should be perfectly uniform on the fracture, homogeneous, very dense, very hard, tough, and reasonably free from "kiln-marks," or indentations made by overlying brick.

Kiln-marks are a splendid guide that the brick have received sufficient heat to vitrify them, and the greater the depth of the kiln-mark the more thoroughly the brick is usually vitrified; but if too deep they make a rough, uneven pavement. There is usually a limit as to what is allowable for the depth of the kiln-mark, which is a matter of opinion for the engineer, and is placed at $\frac{1}{8}$ to $\frac{3}{8}$ inch. Except in fire-clays, it is seldom that a properly vitrified brick is entirely free from slight indentations, unless from the very top of the kiln; with the exception of this one place, a total absence of such marks is apt to indicate underburning.

CHAPTER XIII.

TIMBER.*

CHARACTERISTICS AND PROPERTIES OF WOOD.

178. Structure and Appearance.—The structure of wood affords the only reliable means of distinguishing the different kinds. Color, weight, smell, and other appearances, which are often direct or indirect results of structure, may be helpful in this distinction, but cannot be relied upon entirely. In addition, structure underlies nearly all the technical properties of this important product and furnishes an explanation why one piece differs as to these properties from another.

Structure explains why oak is heavier, stronger, and tougher than pine; why it is harder to saw and plane, and why it is so much more difficult to season without injury. From its less porous structure alone, it is evident that a piece of a young and thrifty oak is stronger than the porous wood of an old or stunted tree; or that Geórgia or long-leaf pine excels white pine in weight and strength. Keeping especially in mind the arrangement and direction of the fibres of wood, it is clear at once why knots and "cross-grains" interfere with the strength of timber.

It is due to structural peculiarities that "honeycombing" occurs in rapid seasoning, that "checks" or cracks extend radially and follow pith-rays, that tangent or "bastard" boards shrink and warp more than quartered lumber. These same peculiarities enable cherry and oak to take a better finish than basswood or coarse-grained pine.

Moreover, structure, aided by color, determines the beauty of wood. All the pleasing figures, whether in a hard-pine ceiling, a desk of quartered oak, or in the beautiful panels of "curly" or "bird's-eye" maple decorating the saloon of a ship or a palace-car, are due to differences in the structure of the wood. Knowing this, the appearance of any particular

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* This chapter is taken from Bulletin 10 of the U. S. Forestry Division, Agricultural Department, 1895; B. E. Fernow, Chief of the Division. The matter contained in this bulletin is mostly the result of original studies made by Mr. Filibert Roth, this work being one department of the "U. S. Timber Investigations."

section can be foretold, and almost unlimited choice and combination are thereby suggested.

Thus a knowledge of structure not only enables us to distinguish the different woods, judge as to their qualities, and explain the causes of their beauty, but it also becomes an invaluable aid to the thoughtful worker, guiding him to a more careful selection and a more perfect use of his material.

179. Classes of Trees.—The timber of the United States is furnished by three well-defined classes of trees: the needle-leaved, naked-seeded conifers (pine, cedar, etc.); the dicotyledonous (with two seed-leaves), broad-leaved trees (oak, poplar, etc.) ; and to an inferior extent by the monocotyledonous (with one seed-leaf), palms, yuccas, and their allies, which last are confined to the most southern parts of the country.

Broad-leaved trees are also known as deciduous trees, although, especially in warm countries, many of them are evergreen,* while the conifers are commonly termed "evergreens," although the larch, bald cypress, and others shed their leaves every fall, and even the names "broad-leaved" and "coniferous," though perhaps the most satisfactory, are not at all exact, for the conifer ginkgo has broad leaves and bears no cones.

In the lumber trade, the woods of broad-leaved trees are known as "hardwoods," though poplar is as soft as pine, and the coniferous woods are "soft woods," notwithstanding that yew ranks high in hardness even when compared to "hardwoods."

Both in the number of different kinds of trees or species and still more in the importance of their product the conifers and broad-leaved trees far excel the palms and their relatives.

In the manner of growth both conifers and broad-leaved trees behave alike, adding each year a new layer of wood which covers the old wood in all parts of the stem and limbs. Thus the trunk continues to grow in thickness throughout the life of the tree by additions (annual rings) which in temperate climates are, barring accidents, accurate records of the tree. With the palms and their relatives the stem remains generally of the same diameter, the tree of a hundred years being no thicker than it was at ten years, the growth of these being only at the top. Even where a peripheral increase takes place, as in the yuccas, the wood is not laid on in well-defined layers; the structure remains irregular throughout.

Though alike in their manner of growth, and therefore similar in their general make-up, conifers and broad-leaved trees differ markedly in the details of their structure and the character of their wood. The wood of all conifers is very simple in its structure, the fibres composing the main part of the wood being all alike and their arrangement regular. The wood of broad-leaved trees is complex in structure; it is made up of several differ-

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* In Ceylon even the cultivated cherry has become an evergreen.

ent kinds of cells and fibres and lacks the regularity of arrangement so noticeable in the conifers. This difference is so great that in a study of wood structure it is best to consider the two kinds separately.

180. Sapwood and Heartwood.—Examining a smooth cross-section or end face of a well-grown log of Georgia pine or Norway pine, we distinguish an envelope of reddish, scaly bark, a small whitish pith at the centre, and between these the wood in a great number of concentric rings.

A zone of wood next to the bark, 1 to 3 or more inches wide, and containing thirty to fifty or more annual rings, is of lighter color; this is the sapwood, the inner, darker part of the log being the heartwood. In the former many cells are active and store up starch and otherwise assist in the life-processes of the tree, although only the last or outer layer of cells (the cambium layer) forms the growing part and the true life of the tree. In the heartwood all cells are lifeless cases, and serve only the mechanical function of keeping the tree from breaking under its own great weight, or from being broken by the winds.

The darker color of the heartwood is due to infiltration of chemical substances into the cell-walls, but the cavities of the cells in pine are not filled up, as is sometimes believed, nor do their walls grow thicker, nor is their wall any more lignified than in the sapwood. Sapwood varies in width and in the number of rings which it contains, even in different parts of the same tree; the same year's growth which is sapwood in one part of a disk may be heartwood in another. Sapwood is widest in the main part of the stem and varies often within considerable limits, and without apparent regularity. Generally it becomes narrower toward the top and in the limbs, its width varying with the diameter, and being least, in a given disk, on the side which has the shortest radius. Sapwood of old and stunted pines is composed of more rings than that of young and thrifty specimens. Thus in a pine two hundred and fifty years old, a layer of wood or annual ring does not change from sapwood to heartwood until seventy or eighty years after it is formed, while in a tree one hundred years old, or less, it remains sapwood only from thirty to sixty years. The width of the sapwood varies considerably for different kinds of pines; it is small for long-leaf and white pine, and great for loblolly and Norway pines. Occupying the peripheral part of the trunk, the proportion which it forms of the entire mass of the stem is always great. Thus even in old trees of long-leaf pine the sapwood forms about 40 per cent of the merchantable log, while in the loblolly and in all young trees the bulk of the wood is sapwood.

181. The Annual Rings.—The concentric, annual, or yearly, rings which appear on the end face of a log are cross-sections of so many thin layers of wood. Each such layer forms an envelope around its inner neighbor, and is in turn covered by the adjoining layer without, so that the whole stem is built up of a series of thin hollow cylinders, or rather cones. A new layer of wood is formed each season, covering the entire

stem, as well as all the living branches. The thickness of this layer, or the width of the yearly ring, varies greatly in different trees and also in different parts of the same tree. In a normally grown, thrifty pine log the rings are widest near the pith, growing more and more narrow toward the bark. Thus the central twenty rings in a disk of an old long-leaf pine may each be one-eighth to one-sixth inch (3 to 4 mm.) wide, while the twenty rings next to the bark may average only one-thirtieth inch (0.8 mm.). In our forest trees rings of one-half inch in width occur only near the centre in disks of very thrifty trees of both conifers and hard woods; one-twelfth inch represents good thrifty growth, and the minimum width of about one two-hundredths inch (0.12 mm.) is often seen in stunted spruce and pine. The average width of rings in well-grown old white pine will vary from one-twelfth to one-eighteenth inch, while in the slower growing long-leaf pine it may be one twenty-fifth to one fiftieth of an inch. The same layer of wood is widest near the stump in very thrifty young trees, especially if grown in the open part, but in old forest trees the same year's growth is wider in the upper part of the tree, being narrowest near the stump and often also near the very tip of the stem. Generally the rings are widest near the centre, growing narrower towards the bark. In logs from stunted trees the order is often reversed, the interior rings being thin and the outer rings widest. Frequently, too, zones or bands of very narrow rings, representing unfavorable periods of growth, disturb the general regularity. Few trees, even among pines, furnish logs with truly circular cross-sections; usually they are oval, and at the stump commonly quite irregular in figure. Moreover, even in very regular or circular disks the pith is rarely in the centre, and frequently one radius is conspicuously longer than its opposite, the width of some of the rings, if not all, being greater on one side than on the other. This is nearly always so in the limbs, the lower radius exceeding the upper.

In extreme cases, especially in the limbs, a ring is frequently conspicuous on one side and almost or entirely lost to view on the other. Where the rings are extremely narrow, the dark portion of the ring is often wanting, the color being quite uniform and light. The greater regularity or irregularity of the annual rings has much to do with the technical qualities of the timber.

182. Spring and Summer Wood (*Coniferous Trees*).—Examining the rings more closely, it is noticed that each ring is made up of an inner, softer, light-colored, and an outer, or peripheral, firmer and darker-colored portion. Being formed in the fore part of the season, the inner, light-colored part is termed spring wood, the outer, darker portion being the summer wood of the ring. Since the latter is very heavy and firm, it determines to a large extent the weight and strength of the wood, and as its darker color influences the shade of color of the entire piece of wood, this color effect becomes a valuable aid in distinguishing heavy and strong

from light and soft pine wood. In most hard pines, like the long-leaf, the dark summer wood appears as a distinct band, so that the yearly ring is composed of two sharply defined bands—an inner, the spring wood, and an outer, the summer wood. But in some cases, even in hard pines, and normally in the wood of white pines, the spring wood passes gradually into the darker summer wood, so that a sharply defined line occurs only where the spring wood of one ring abuts against the summer wood of the previous year's growth. It is this clearly defined line which enables the eye to distinguish even the very narrow rings in old pines and spruces. In some cases, especially in the trunks of Southern pines, and normally on the lower side of pine limbs, there occur dark bands of wood in the spring-wood portion of the ring, giving rise to false rings which mislead in a superficial counting of rings. In the disks cut from limbs these dark bands often occupy the greater part of the ring and appear as "lunes" or sickle-shaped figures. The wood of these dark bands is similar to that of the true summer wood—the cells have thick walls, but usually lack the compressed or flattened form.

Normally, the summer wood forms a greater proportion of the ring in the part of the tree formed during the period of thriftiest growth. In an old tree this proportion is very small in the first two to five rings about the pith, and also in the part next to the bark, the intermediate part showing a greater proportion of summer wood. It is also greatest in a disk taken from near the stump and decreases upward in the stem, thus fully accounting for the difference in weight and firmness of the wood of these different parts. In the long-leaf pine the more substantial summer wood often forms scarcely 10 per cent of the wood in the central five rings; 40 to 50 per cent of the next one hundred rings; about 30 per cent in the next fifty, and only about 20 per cent in the fifty rings next to the bark. It averages 45 per cent of the wood of the stump and only 24 per cent of that of the top.

Sawing the log into boards, the yearly rings are represented on the faces of the middle board (radial sections) by narrow, parallel stripes (see Fig. 84), an inner, lighter stripe, and its outer, darker neighbor, always corresponding to one annual ring.

On the faces of the boards nearest the slab (tangential or "bastard" boards) the several years' growth should also appear as parallel but much broader stripes. This they do only if the log is short and very perfect. Usually a variety of pleasing patterns is displayed on the boards, depending on the position of the saw-cut and on the regularity of growth of the log. (See Fig. 84.)

Where the cut passes through a prominence (bump or crook) of the log, irregular, concentric circlets and ovals are produced, and on almost all tangent boards V-shaped forms occur.

183. Anatomical Structure of Coniferous Woods.—Holding a well-smoothed disk or cross-section one-eighth inch thick toward the light, it is

readily seen that pine wood is a very porous structure. If viewed with a strong magnifier, the little tubes, especially in the spring-wood of the rings, are easily distinguished and their arrangement in regular straight radial rows is apparent. Scattered through the summer-wood portion of the rings, numerous irregular grayish dots (the resin-ducts) disturb the uniformity and regularity of the structure. Magnified one hundred times, a piece of spruce, which is similar to pine, presents a picture like that shown in Fig. 85. Only short pieces of the tubes or cells of which the wood is composed are represented in the picture.

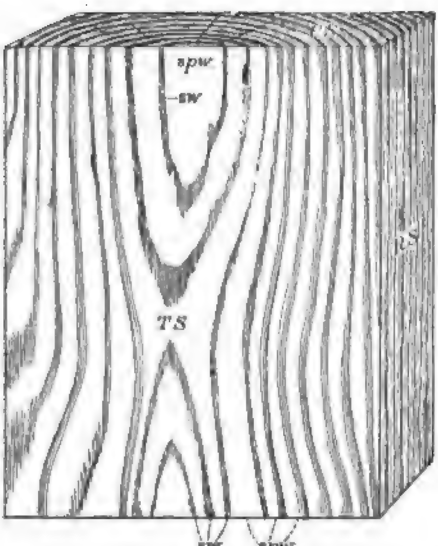

FIG. 84.—Board of Pine. *CS*, cross-section; *RS*, radial section; *TS*, tangential section; *sw*, summer wood; *spw*, spring wood.*

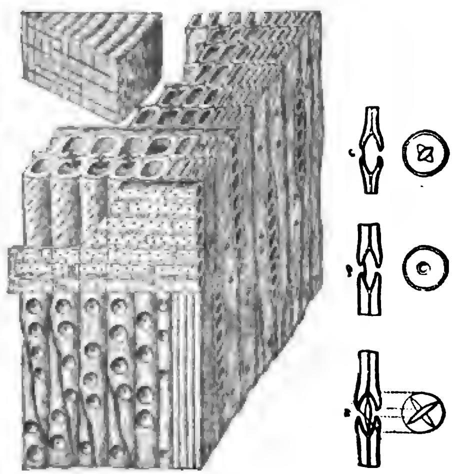

FIG. 85.—Wood of Spruce. 1, natural size; 2, small part of one ring magnified 100 times. The vertical tubes are wood-fibres, in this case all "tracheids." *m*, medullary or pith ray; *n*, transverse tracheids or pith-ray; *a*, *b*, and *c*, bordered pits of the tracheids, more enlarged.

The total length of these fibres is one-twentieth to one-fifth inch, being smallest near the pith, and is fifty to one hundred times as great as their width (Fig. 86). They are tapered and closed at their ends, polygonal or rounded and thin-walled, with a large cavity (lumen) or internal space in the spring wood, while they are thick-walled and flattened radially, with the internal space or lumen much reduced, in the summer wood. (See right-hand portion of Fig. 85.) This flattening, together with the thicker walls of the cells which reduces the lumen, produces the greater firmness and darker

* This figure is deceptive inasmuch as the more open or porous spring wood is represented by a plain white surface, as though it were solid, while the more solid summer wood is represented by a shaded surface as though it were more porous. The reverse is of course the case.—J. B. J.

Fig. 86.—Group of Fibres from Pine Wood. Partly schematic. The little circles are "bordered pits" (see Fig. 85, a–c). The transverse rows of square pits indicate the places of contact of these fibres and the cells of the neighboring pith-rays. Magnified about 50 times.

color of the summer wood; that is to say, there is more material in the same volume. As shown in the figure, the tubes, cells, or "tracheids" are decorated on their walls by circlet-like structures, called "bordered pits," sections of which are seen more magnified at *a*, *b*, and *c*, Fig. 85. These pits are in the nature of pores, covered by very thin membranes, and serve as waterways between the cells or tracheids.

The dark lines on the side of the smaller piece (1, Fig. 85) appear when magnified (in 2, Fig. 85) as tiers of eight to ten rows of cells, lying in vertical radial planes, and are seen as bands on the radial face, and as rows of pores on the tagential face. These bands or tiers of cell-rows are the "medullary rays" or "pith-rays," and are common to all our lumber woods. In the pines and other conifers they are quite small, but they can readily be seen, even without a magnifier, if a radial surface of split wood (not smooth) is examined. The entire radial face will be seen almost covered with these tiny structures, which appear as fine but conspicuous cross-lines. As shown in Fig. 85, the cells of the medullary or pith rays are smaller and very much shorter than the wood-fibres or tracheids, see *b*, Fig. 90, and their long axis is at right angles to that of the fibres. In pines and spruces the cells of the upper and lower rows of each tier or pith-ray have "bordered" pits like those of the wood-fibres or tracheids proper, but the cells of the intermediate rows, and of all rows in the rays of cedars, etc., have only "simple" pits, i.e., pits devoid of the saucer-like "border" or rim.

In pine, many of the pith-rays are larger than the

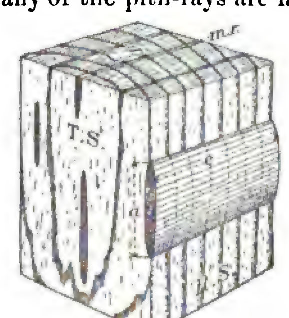

Fig. 87.—Block of Oak. *C.S.*, cross-section; *R.S.*, radial section; *T.S.*, tangential section; *m.r.*, medullary or pith ray; *a*, height, *b*, width, and *c*, length of a pith-ray.

majority, each containing a whitish line, the horizontal resin-duct, which, though much smaller, resembles the vertical ducts seen on the cross-section. The larger vertical resin-ducts * are best observed on the removal of the bark from a fresh piece of white pine, cut in winter, where they appear as conspicuous white lines, extending often for many inches up and down the stem.

Neither the horizontal nor the vertical resin-ducts are vessels or cells, but are openings between cells, i.e., intercellular spaces in which the resin accumulates, freely oozing out when the ducts of a fresh piece of sapwood are cut. They are present only in our coniferous woods, and even here they are restricted to pine, spruce, and larch, and are normally absent in fir, cedar, cypress, and yew.

Altogether the structure of coniferous wood is very simple and regular, the bulk being made up of small fibres called tracheids, the disturbing elements of pith-rays and resin-ducts being insignificant, and hence the great uniformity and great technical value of coniferous wood.

184. Anatomical Structure of Broad-leaved Trees.—On a cross-section of oak, the same arrangement of pith and bark, of sapwood and heartwood, and the same disposition of the wood in well-defined concentric or annual rings occurs, but the rings are marked by lines, or rows, of conspicuous pores or openings which occupy the greater part of the spring wood of each ring (see Fig. 87, also Fig. 89) and are, in fact, the openings through the vessels cut by the section. On the radial section, or quarter-sawed board, the several layers appear as so many parallel stripes (see Fig. 88); on the tangential section or "bastard" face, patterns similar to those mentioned for pine wood are observed. But while the patterns in hard pine are marked by the darker summer wood and are composed of plain, alternating stripes of darker and lighter wood, the figures in oak (and other broad-leaved woods) are due chiefly to the vessels, those of the spring wood in oak being the most

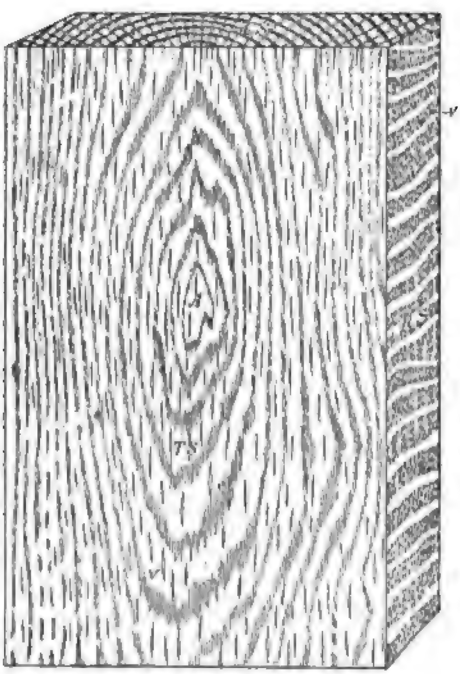

FIG. 88.—Board of Oak. *CS*, cross-section; *RS*, radial section; *TS*, tangential section; *v*, vessels or pores, cut through; *A*, slight curve in log which appears in section as an islet.

* See *rd*, Fig. 118.

conspicuous (see Fig. 88); so that in an oak table the darker, shaded parts are the spring wood, the lighter parts the summer wood.

On closer examination of the smoothed cross-section of oak, the spring-wood part of the ring is found to be formed, in great part, of pores: large, round, or oval openings through long vessels. These are separated by a

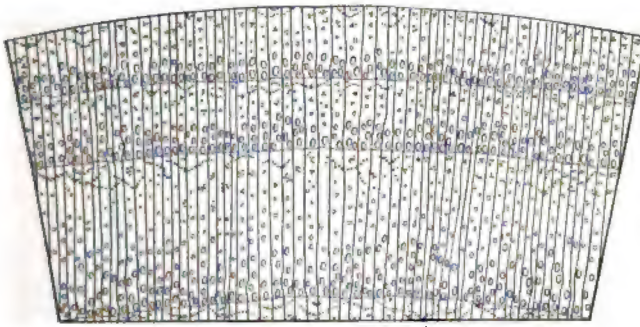

FIG. 89.—Cross-section of Oak magnified about 5 times.

grayish and quite porous tissue (see Fig. 89) which continues here and there in the form of radial, often branched, patches (not the pith-rays) into and through the summer wood to the spring wood of the next ring. The large vessels of the spring wood, occupying 6 to 10 per cent of the volume of a log in very good oak, and 25 per cent or more in inferior and narrow-ringed lumber, are a very important feature, since it is evident that the greater their share in the volume, the lighter and weaker the wood. They are smallest near the pith, and grow wider outward; they are wider in the stem than limb and seem to be of indefinite length, forming open channels in some cases probably as long as the tree itself.

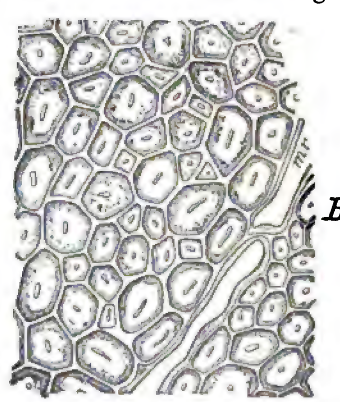

FIG. 90.—Portion of the Firm Bodies of Fibres with Two Cells of a small Pith-ray, *mr*. Highly magnified.

Scattered through the radiating gray patches of porous wood are vessels similar to those of the spring wood, but decidedly smaller. These vessels are usually fewer and larger near the spring wood, and smaller and more numerous in the outer portions of the ring. Their number and size can be utilized to distinguish the oaks classed as white oaks from those classed as black and red oaks; they are fewer and larger in red oaks, smaller but much more numerous in white oaks. The summer wood, except for these radial grayish patches, is dark-colored and firm. This firm portion, divided into bodies or strands by these patches of porous wood and also by fine wavy concentric lines of short, thin-walled cells (see Fig. 89), consists of thick-walled fibres (see Fig. 90) and is the chief element of

strength in oak wood. In good white oak it forms one half and more of the wood; it cuts like horn, and the cut surface is shiny and of a deep chocolate-brown color. In very narrow-ringed wood and in inferior red oak it is usually much reduced in quantity as well as quality.

The pith-rays of the oak, unlike those of coniferous woods, are at least in part very large and conspicuous (see Fig. 87, their height indicated by the letter *a*, and their width by the letter *b*). The large medullary rays of oak are often twenty and more cells thick and several hundred cell-rows in height, which amount commonly to one or more inches. These large rays are conspicuous on all sections. They appear as long, sharp, grayish lines on the cross-section, as short, thick lines, tapering at each end, on the tangential or "bastard" face, and as broad, shiny bands, the "silver grain" or "mirrors," on the radial section. In addition to these coarse rays, there is also a large number of small pith-rays, which can be seen only when magnified. On the whole, the pith-rays form a much larger part of the wood than might be supposed. In specimens of good white oak it has been found that they formed about 16 to 25 per cent of the wood.

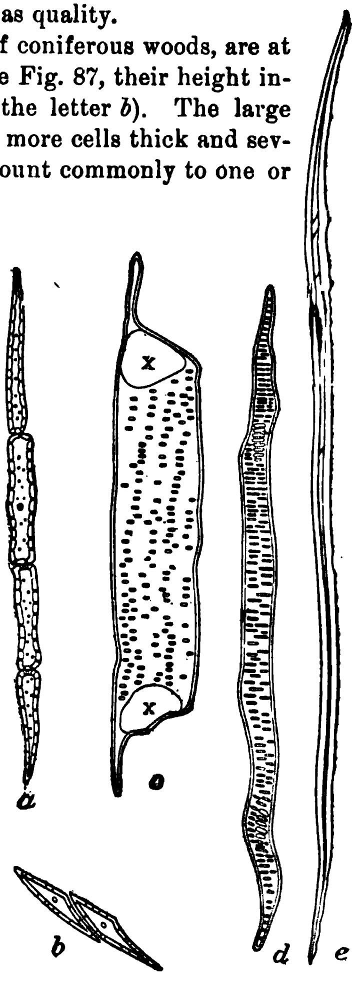

FIG. 91.—Isolated Fibres and Cells. *a*, four cells of wood-parenchyma; *b*, two cells from a pith-ray; *c*, a single joint or cell of a vessel, the openings *x* leading into its upper and lower neighbors; *d*, tracheid; *e*, wood-fibre proper.

185. Minute Structure. — If a well-smoothed, thin disk or cross-section of oak (say one-sixteenth inch thick) is held up to the light, it looks very much like a sieve, the pores or vessels appearing as clean-cut holes; the spring wood and gray patches are seen to be quite porous, but the firm bodies of fibres between them are dense and opaque. Examined with the magnifier it will be noticed that there is no such regularity of arrangement in straight rows as is conspicuous in the pine; on the contrary, great irregularity prevails. At the same time, while the pores are as as large as pin-holes, the cells of the denser wood, unlike those of pine wood, are too small to be distinguished. Studied with the microscope, each vessel is found to be a vertical row of a great number of short, wide tubes, joined end to end (Fig. 91, *c*). The porous spring wood and radial gray tracts are partly composed of smaller vessels,

but chiefly of tracheids like those of pine, and of shorter cells, the "wood-parenchyma," resembling the cells of the medullary rays. These latter, as well as the fine concentric lines mentioned as occurring in the summer wood, are composed entirely of short, tube-like parenchyma-cells with square or oblique ends (Fig. 91, *a* and *b*). The wood-fibres proper, which form the dark, firm bodies referred to, are very fine, thread-like cells one twenty-fifth to one-tenth inch long, with a wall commonly so thick that scarcely any empty internal space or lumen remains (Figs. 91, *e*, and 90).

If instead of oak a piece of poplar or basswood (Fig. 92) had been used in this study, the structure would have been found to be quite different.

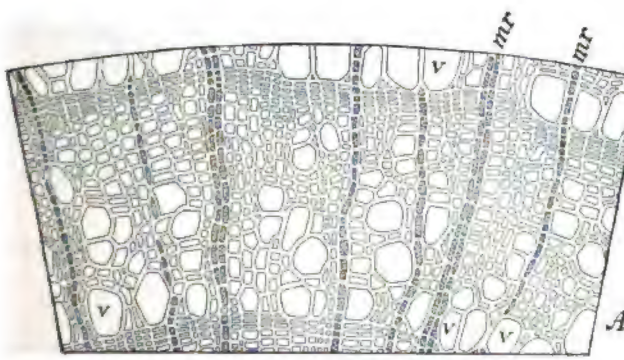

Fig. 92—Cross-section of Basswood (magnified). *v*, vessels; *mr*, pith-rays.

The same kinds of cell-elements, vessels, etc., are present, but their combination and arrangement is different, and thus from the great variety of possible combinations results the great variety of structure and, in consequence, of the qualities which distinguish the wood of broad-leaved trees. The sharp distinction of sapwood and heartwood is wanting; the rings are not so clearly defined, the vessels of the wood are small, very numerous, and rather evenly scattered through the wood of the annual ring, so that the distinction of the ring almost vanishes and the medullary or pith rays, in poplar, can be seen, without being magnified, only on the radial section.

186. Different "Grains" of Wood.—The terms "fine-grained," "coarse-grained," "straight-grained," and "cross-grained" are frequently applied in woodworking. In common usage, wood is "coarse-grained" if its annual rings are wide, "fine-grained" if they are narrow; in the finer wood industries a "fine-grained" wood is capable of high polish, while a "coarse-grained" wood is not, so that in this latter case the distinction depends chiefly on hardness, and in the former on an accidental case of slow or rapid growth.

Generally the direction of the wood-fibres is parallel to the axis of the stem or limb in which they occur, the wood is straight-grained, but in many cases the course of the fibres is spiral or twisted around the tree as

shown in Fig. 93, and sometimes (commonly in butts of gum and cypress) the fibres of several layers are oblique in one direction, and those of the

Fig. 93.

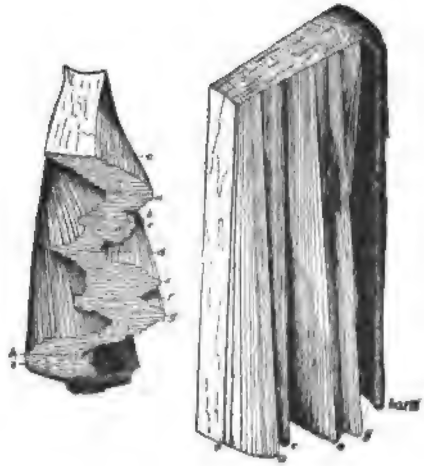

Fig. 94.

Fig. 93.—Spiral Grain. Season-checks, after removal of bark, indicate the direction of the fibres or grain.

Fig. 94.—Alternating Spiral Grain in Cypress. Side and end view of same piece. When the bark was at *o* the grain at this point was straight. From that time each year it grew more oblique in one direction, reaching a climax at *a*, and then turned back in the opposite direction. These alternations were repeated periodically, the bark sharing in these changes.

next series of layers are oblique in the opposite direction, as shown in Fig. 94; the wood is cross- or twisted-grained. Wavy grain in a tangential plane as seen on the radial section is illustrated in Fig. 94*a*, which represents an extreme case observed in beech. This same form also occurs on the radial plane, causing the tangential section to appear wavy or in transverse folds. When wavy grain is fine, i.e., the folds or ridges small but numerous, it gives rise to the "curly" structure frequently seen in maple. Ordinarily, neither wavy, spiral, nor alternate grain is visible on the cross-section; its existence often escapes the eye even on smooth, longitudinal faces in sawed material, so that the only safe guide to their discovery lies in splitting the wood in the two normal planes.

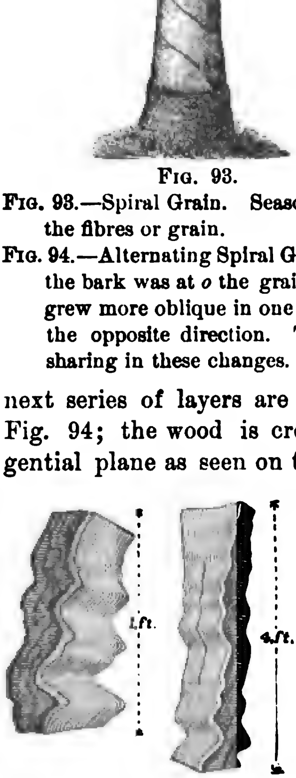

Fig. 94*a*. — Wavy Grain in Beech. (After Nördlinger.)

Generally the surface of the wood under the bark, and therefore also that of any layer in the interior, is not uniform and smooth, but is channelled and pitted by numerous depressions which differ greatly in size and form. Usually, any one depression or elevation is restricted to one or a few annual layers (i.e., seen only in one or a few rings), and is then lost,

being compensated (the surface at the particular spot evened up) by growth. In some woods, however, any depression or elevation once attained grows from year to year and reaches a maximum size which is maintained for many years, sometimes throughout life.

In maple, where this tendency to preserve any particular contour is very great, the depressions and elevations are usually small (commonly less than one-eighth inch, but very numerous. On tangent boards of such wood the sections of these pits and prominences appear as circlets and give rise to the beautiful "bird's-eye" or "landscape" structure. Similar structures in the burls of black ash, maple, etc., are frequently due to the presence of dormant buds, which cause the surface of all the layers through which they pass to be covered by small conical elevations, whose cross-sections on the sawed board appear as irregular circlets or islets each with a dark speck, the section of the pith or "trace" of the dormant bud in the centre.

In the wood of many broad-leaved trees the wood-fibres are much longer when full grown than when they are first formed in the cambium or growing zone. This causes the tips of each fibre to crowd in between the fibres above and below, and leads to an irregular interlacement of these fibres, which adds to the toughness but reduces the cleavability of the wood.

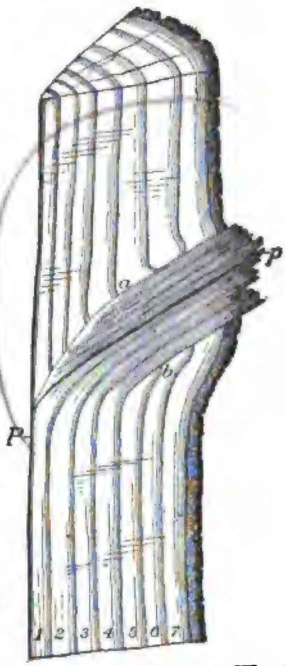

FIG. 95.—Section of Wood showing Position of the Grain at Base of a Limb which has been Dead Three Years. *P*, pith of both stem and limb; 1–7, seven yearly layers of wood; *a*, *b*, knot or basal part of a limb which lived four years, then died and broke off near the stem, leaving the part to the left of *a*, *b*, a "sound" knot, the part to the right a "dead" knot, which would soon be entirely covered by the growing stem.

At the junction of limb and stem the fibres on the upper and lower sides of the limb behave differently. On the lower side they run from the stem into the limb, forming an uninterrupted strand or tissue and a perfect union. On the upper side the fibres bend aside, are not continuous into the limb, and hence the connection is imperfect (Fig. 95).

Owing to this arrangement of the fibres, the cleft made in splitting never runs into the knot, if started on the side above the limb, but is apt to enter the knot if started below, a fact well understood in woodcraft. When limbs die, decay, and break off, the remaining stubs are surrounded and may finally be covered by the growth of the trunk, and thus give rise to the annoying "dead" or "loose" knots.

187. Color and Odor.—Color, like structure, lends beauty to the wood,

aids in its identification, and is of great value in the determination of its quality. Considering only the heartwood, the black color of the persimmon, the dark brown of the walnut, the light brown of the white oaks, the reddish brown of the red oaks, the yellowish white of the tulip and poplar, the brownish red of the redwood and cedar, the yellow of the papaw and sumac, are all reliable marks of distinction; and color together with lustre and weight are only too often the only features depended upon in practice. Newly formed wood, like that of the outer few rings, has but little color. The sapwood generally is light, and the wood of trees which form no heartwood changes but little, except when stained by forerunners of disease.

The different tints of colors, whether the brown of oak, the orange-brown of pine, the blackish tint of walnut, or the reddish cast of cedar, are due to pigments, while the deeper shade of the summer-wood bands in pine and cedar, or in oak or walnut, is due to the fact that, the wood being denser, more of the colored wood substance occurs on a given space, i.e., there is more colored matter per square inch.

Wood is translucent, a thin disk of pine permitting light to pass through quite freely. This translucency affects the lustre and brightness of lumber. When wood is attacked by fungi it becomes more opaque, loses its brightness, and in practice is designated "dead" in distinction from "live" or bright timber. Exposure to air darkens all wood; direct sunlight and occasional moistening hasten this change and cause it to penetrate deeper. Prolonged immersion has the same effect, pine wood becoming a dark gray, while oak changes to a blackish brown.

Odor, like color, depends on chemical compounds, forming no part of the wood substance itself. Exposure to weather reduces and often changes the odor, but a piece of dry long-leaf pine, cedar, or camphor wood exhales apparently as much odor as ever when a new surface is exposed.

Heartwood is more odoriferous than sapwood. Many kinds of wood are distinguished by strong and peculiar odors. This is especially the case with camphor, cedar, pine, oak, and mahogany, and the list would comprise every kind of wood in use were our sense of smell developed in keeping with its importance. Decomposition is usually accompanied by pronounced odors; decaying poplar emits a disagreeable odor, while red oak often becomes fragrant, its smell resembling that of heliotrope.

188. Resonance.—If a log or scantling is struck with the axe or hammer, a sound is emitted which varies in pitch and character with the shape and size of the stick, and also with the kind and condition of wood. Not only can sound be produced by a direct blow, but a thin board may be set vibrating and be made to give a tone by merely producing a suitable tone in its vicinity. The vibrations of the air, caused by the motion of the strings of the piano, communicate themselves to the board, which vibrates in the same intervals as the string and reenforces the note. The note which a

given piece of wood may emit varies in pitch directly with the elasticity, and indirectly with the weight, of the wood. The ability of a properly shaped sounding-board to respond freely to all the notes within the range of an instrument, as well as to reflect the character of the notes thus emitted (i.e., whether melodious or not), depends, first, on the structure of the wood and next on the uniformity of the same throughout the board. In the manufacture of musical instruments all wood containing defects, knots, cross grain, resinous tracts, alternations of wide and narrow rings, and all wood in which summer and spring wood are strongly contrasted in structure and variable in their proportions, is rejected, and only radial sections (quarter-sawed or split) of wood of uniform structure and growth are used.

The irregularity in structure, due to the presence of relatively large pores and pith-rays, excludes almost all our broad-leaved woods from such use, while the number of eligible woods among conifers is limited by the necessity of combining sufficient strength with uniformity in structure, absence of two pronounced bands of summer wood, and relative freedom from resin.

Spruce is the favored resonance wood; it is used for sounding-boards both in pianos and violins, while for the resistant back and sides of the latter the highly elastic hard maple is used. Preferably resonance wood is not bent to assume the final form; the belly of the violin is shaped from a thicker piece, so that every fibre is as nearly in its original unstrained condition as possible, and therefore free to vibrate. All wood for musical instruments is, of course, well seasoned, the final drying in kiln or warm room being preceded by careful seasoning at ordinary temperatures often for as many as seven years or more. The improvement of violins, not by age but by long usage, is probably due not only to the adjustment of the numerous component parts to each other, but also to a change in the wood itself; years of vibrating enabling any given part to vibrate much more readily.

SPECIFIC GRAVITY, OR WEIGHT.

189. Weight Dependent on Structure and Moisture.—A small cross-section of wood, as in Fig. 96, dropped into water, sinks, showing that the substance of which wood-fibre or wood is built up is heavier than water. By immersing the wood successively in heavier liquids, until we find a liquid in which it does not sink, and comparing the weight of the same with water, we find that wood-substance is about 1.6 times as heavy as water, and that this is as true of poplar as of oak or pine.

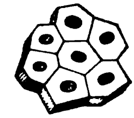

FIG. 96. Cross-section of a Group of Wood-fibres.

Separating a single cell, as shown in Fig. 97, *a*, drying and then dropping it into water, it floats. The air-filled cell-cavity or interior reduces its weight, and, like a corked empty bottle, it weighs less than the water. Soon, however, water soaks into the cell, when it fills up and sinks.

Many such cells grown together, as in a block of wood, sink when all or most of them are filled with water, but will float as long as the majority are empty or only partly filled. This is why a green, sappy pine pole soon sinks in "driving" (floating). Its cells are largely filled before it is thrown in, and but little additional water suffices to make its weight greater than that of the water.

In a white-pine log, composed chiefly of empty cells (heartwood), the water requires a very long time to fill up the cells (five years would not suffice to fill them all), and therefore the log may float for many months or even years. When the wall of the wood-fibre is very thick (five-eighths or more of the volume), as in Fig. 97, *b*, the fibre sinks whether empty or filled. This applies to most of the fibres of the dark summer-wood bands in pines, and to the compact fibres of oak or hickory, and many, especially tropical woods, have such thick-walled cells and so little empty or air space that they never float.

Here, then, are the two main factors of weight in wood: The amount of cell-wall, or wood-substance, constant for any given piece, and the amount of water contained in the wood, variable even in the standing tree, and only in part eliminated in drying.

The weight of the green wood of any species varies chiefly as the second factor, and is entirely misleading if the relative weight of different kinds is sought. Thus some green sticks of the otherwise lighter cypress and gum sink more readily than fresh oak.

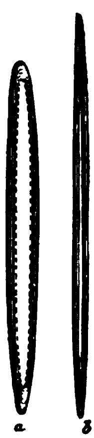

FIG. 97.—Isolated Fibres.

The weight of sapwood, or the sappy peripheral part of our common lumber woods, is always great whether cut in winter or summer. It rarely falls much below 45 pounds and commonly exceeds 55 pounds to the cubic foot, even in our lighter wooded species.

It follows that the green wood of a sapling is heavier than that of an old tree, the fresh wood from a disk of the upper part of a tree often heavier than that of the lower part, and the wood near the bark heavier than that nearer the pith, and also that the advantage of drying the wood before shipping is most important in sappy and light kinds.

When kiln-dried, the misleading moisture factor of weight is uniformly reduced and a fair comparison made possible. For the sake of convenience in comparison, the weight of wood is expressed either as the weight per cubic foot or, what is still more convenient, as specific weight or density.

190. Variation in Weight in a Single Trunk.—If an old long-leaf pine is cut up as shown in Fig. 98, the wood of disk No. 1 is heavier than that of disk No. 2, the latter heavier than that of disk No. 3, and the wood of the top disk is found to be only about three fourths as heavy as that of disk No. 1.

Similarly, if disk No. 2 is cut up as in the figure, the specific weight of the different pieces is:

a about 0.52
b about 0.64
c about 0.67
d, e, f about 0.65

Showing that in this disk, at least, the wood formed during the many years' growth, represented in piece *a*, is much lighter than that of former years. It also shows that the best wood is the middle part, with its large proportion of dark summer-wood bands.

Cutting up all disks in the same way, it will be found that the piece *a* of the first disk is heavier than piece *a* of the fifth, and that piece *c* of the first disk excels the piece *c* of all the other disks. This shows that the wood grown during the same number of years is lighter in the upper parts of the stem; and if the disks are smoothed on their radial surfaces and set up one on top of the other in their regular order for sake of comparison, this decrease in weight will be seen to bè accompanied by a decrease in the amount of summer wood. The color effect of the upper disks is conspicuously lighter.

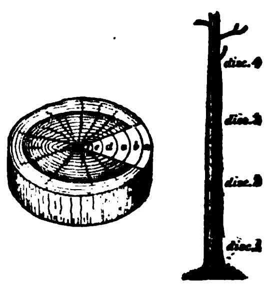

FIG. 98.—Orientation of Wood Samples.

If our old pine had been cut one hundred and fifty years ago, before the outer, lighter wood was laid on, it is evident that the weight of the wood of any one disk would have been found to increase from the centre outward, and no subsequent decrease could have been observed.

In a thrifty young pine, then, the wood is heavier from the centre outward, and lighter from below upward; only the wood laid on in old age falls in weight below the average. The number of brownish bands of summer wood are a direct indication of these differences.

If an old oak is cut up in the same manner, the butt cut is also found heaviest and the top lightest, but, unlike the disk of pine, the disk of oak has its firmest wood at the centre, and each successive piece from the centre outward is lighter than its inner neighbor.

Examining the pieces, this difference is not as readily explained by the appearance of each piece as in the case of pine wood. Nevertheless, one conspicuous point appears at once: the pores, so very distinct in oak, are very minute in the wood near the centre and thus the wood is far less porous. Studying different trees it is found that, in the pines, wood with narrow rings is just as heavy as, and often heavier than, the wood with wider rings, but if the rings are unusually narrow in any part of the disk the wood has a lighter color; that is, there is less summer wood and therefore less weight. In oak, ash, or elm trees of thrifty growth, the wider rings (not less than

one-twelfth inch) always form the heaviest wood, while any piece with very narrow rings is light. On the other hand, the weight of a piece of hard maple or birch is quite independent of the width of its rings, since the structure here is uniform across the entire width of the annual ring.

The bases of limbs (knots) are usually heavy, very heavy in conifers, and also the wood which surrounds them, but generally the wood of the limbs is lighter than that of the stem, and the wood of the roots is the lightest.

191. Weight of Different Species.—In general, it may be said that none of the native woods in common use in this country are, when dry, as heavy as water, i.e., 62 pounds to the cubic foot. Few exceed 50 pounds, while most of them fall below 40 pounds, and much of the pine and other coniferous wood weighs less than 30 pounds per cubic foot.

The weight of the wood is, in itself, an important quality. Weight assists in distinguishing maple from poplar. Lightness, coupled with great strength and stiffness, recommends wood for a thousand different uses. *To a large extent weight predicates the strength of the wood, at least in the same species, so that a heavy piece of oak will exceed in strength a light piece of the same species, and in pine it appears probable that, weight for weight, the strength of the wood of various pines is nearly equal, all being reduced to the same degree of dryness.*

WEIGHT OF KILN-DRIED WOOD OF DIFFERENT SPECIES.

Common Name of Species.	Approximate.		
	Specific Gravity.	Weight of—	
		1 cubic foot.	1000 feet of lumber.
(a) Very heavy woods:		Pounds.	Pounds.
* Hickory, oak, persimmon, osage orange, black locust, hackberry, blue beech, best of elm, and ash..........	0.70–0.80	42–48	3700
(b) Heavy woods:			
Ash, elm, cherry, birch, maple, beech, walnut, sour gum, coffee-tree, honey-locust, best of Southern pine, and tamarack..................................	.60– .70	36–42	3200
(c) Woods of medium weight:			
Southern pine, pitch-pine, tamarack, Douglas spruce, Western hemlock, sweet gum, soft maple, sycamore, sassafras, mulberry, light grades of birch and cherry..	.50– .60	30–36	2700
(d) Light woods:			
Norway and bull pine, red cedar, cypress, hemlock, the heavier spruce and fir, redwood, basswood, chestnut, butternut, tulip, catalpa, buckeye, heavier grades of poplar	.40– .50	24–30	2200
(e) Very light woods:			
White pine, spruce, fir, white cedar, poplar............	.30– .40	18–24	1800

* For the scientific names of timbers see the list of Useful American Timbers at the end of this chapter.

Since ordinary lumber contains knots and also more water than is here assumed, and also since its dimensions either exceed or fall short of perfect measurement, the figures in the table are only approximate.

Thus 1000 feet, B. M., of long-leaf pine weighs:

	Pounds.
Rough and green	4500
Boards rough but seasoned	3500
Boards dressed and seasoned	3000
Flooring, matched dressed and seasoned	2500
Weather-boarding, bevelled and dressed	1500

MOISTURE IN WOOD.

192. Moisture Distribution.—Water may occur in wood in three conditions: (1) it forms the greater part (over 90 per cent) of the protoplasmic contents of the living cells; (2) it saturates the walls of all cells; and (3) it entirely or at least partly fills the cavities of the lifeless cells, fibres, and vessels. In the sapwood of pine it occurs in all three forms; in the heartwood only in the second form, that is, it merely saturates the walls. Of 100 pounds of water associated with 100 pounds of dry wood-substance in 200 pounds of fresh sapwood of white pine, about 35 pounds are needed to saturate the cell walls, less than 5 pounds are contained in living cells, and the remaining 60 pounds partly fill the cavities of the wood-fibres. This latter forms the sap as ordinarily understood. It is water brought from the soil, containing small quantities of mineral salts, and in certain species (maple, birch, etc.) it also contains at certain times a small percentage of sugar and other organic matter. These organic substances are the dissolved reserve food stored during winter in the pith-rays, etc., of the wood and bark; generally but a mere trace of them is to be found. From this it appears that the solids contained in the sap, such as albumen, gum, sugar, etc., cannot exercise the influence on the strength of the wood which is so commonly claimed for them.

The wood next to the bark contains the most water. In the species which do not form heartwood the decrease toward the pith is gradual; but where this is formed, the change from a more moist to a drier condition is usually quite abrupt at the sapwood limit. In long-leaf pine the wood of the outer 1 inch of a disk may contain 50 per cent of water, that of the next, or second inch, only 35 per cent, and that of the heartwood only 20 per cent. In such a tree the amount of water in an entire cross-section varies with the amount of sapwood, and is therefore greater for the upper than the lower cuts, greater for limbs than stems, and greatest of all in the roots.

Different trees, even of the same kind and from the same place, differ as to the amount of water they contain. A thrifty tree contains more water

than a stunted one, and a young tree more than an old one, while the wood of all trees varies in its moisture relations with the season of the year.

Contrary to the general belief, a tree contains about as much water in winter as in summer. The fact that the bark peels easily in the spring depends on the presence of incomplete, soft tissue, the rapidly growing cambium layer found between wood and bark during this season, and has little to do with the total amount of water contained in the wood of the stem.

Even in the living tree a flow of sap from a cut occurs only in certain kinds of trees and under special circumstances; from boards, timber, etc., the water does not flow out, as is sometimes believed, but must be evaporated.*

193. Drying Timber.—The rapidity with which water is evaporated, that is, the rate of drying, depends on the size and shape of the piece and on the structure of the wood. An inch board dries more than four times as fast as a 4-inch plank and more than twenty times as fast as a 10-inch timber. White pine dries faster than oak. A very moist piece of pine or oak will, during one hour, lose more than four times as much water per square inch from the cross-section, but only one half as much from the tangential as from the radial section.

In a long timber, where the end or cross-sections form but a small part of the drying surface, this difference is not so evident. Nevertheless, the ends dry and shrink first, and being opposed in this shrinking by the more moist adjoining parts they check, the cracks largely disappearing as seasoning progresses.

High temperatures are very effective in evaporating the water from wood no matter how humid the air. A fresh piece of sapwood may lose weight in boiling water, and can be dried to quite an extent in hot steam.

Kept on a shelf in an ordinary dwelling, wood still retains 8 to 10 per cent of its weight of water, and this percentage is always greater than the percentage of moisture in the surrounding air. Nor is the amount of water in dry wood constant; the weight of a panful of shavings varies with the time of day, being on a summer day greatest in the morning and least in the afternoon.

Desiccating the air with chemicals will cause the wood to dry, but wood thus dried at 80° F. will still lose water in the kiln. Wood dried at 120° F. loses water still if dried at 200° F., and this again will lose more water if the temperature is raised. Absolutely dry wood cannot be obtained; chemical destruction sets in before all the water is driven off.

On removal from the kiln the wood at once takes up water from the air,

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* The seeming exceptions to this rule are mostly referable to two causes, namely: (*a*) Clefts or "shakes" will allow water contained in them to flow out. (*b*) From sound wood, if very sappy, water is forced out whenever the wood is warmed, just as water flows from green wood in the stove.

even in the driest weather. At first the absorption is quite rapid; at the end of a week a short piece of pine, 1½ inches thick, has regained two thirds of, and in a few months all, the moisture which it had when air-dry, 8 to 10 per cent, and also its former dimensions.

In thin boards all parts soon attain the same degree of dryness; in heavy timbers the interior remains moister for many months, and even years, than the exterior parts. Finally an equilibrium is reached, and then only the outer parts change with the weather.

With kiln-dried wood all parts are equally dry, and when exposed the moisture coming from the air must pass in through the outer parts, and thus the order is reversed. Timber seasoned out of doors requires months, or even years, before it is at its best; kiln-dry timber, if properly handled, is prime at once.

Dry wood when soaked in water soon regains its original volume, and in the heartwood portion it may even surpass it; that is to say, swell to a larger dimension than it had when green. With the soaking it continues to increase in weight, the cell-cavities filling with water, and if left many months all pieces sink. Yet even after a year's immersion a piece of oak 2 by 2 inches and only 6 inches long still contains air, i.e., it has not taken up all the water it can. By rafting, or prolonged immersion, wood loses some of its weight, soluble materials being leached out, but it is not impaired either as fuel or as building material. Immersion and, still more, boiling and steaming reduce the hygroscopicity of wood and, therefore, also the troublesome "working," or shrinking and swelling.

Exposure in dry air to a temperature of 300° F. for a short time reduces, but does not destroy, the hygroscopicity and with it the tendency to shrink and swell. A piece of red oak which has been subjected to a temperature of over 300° F. still swells in hot water and shrinks in the kiln.

In artificial drying, temperatures of from 158° to 180° F. are usually employed. Pine, spruce, cypress, cedar, etc., are dried fresh from the saw, allowing four days for 1-inch boards; hard woods, especially oak, ash, maple, birch, sycamore, etc., are air-seasoned for three to six months, to allow the first shrinkage to take place more gradually, and are then exposed to the above temperatures in the kiln for about six to ten days for 1-inch lumber. Freshly cut poplar and cottonwood are often dried directly in kilns.

By employing lower temperatures, 100° to 120° F., green oak, ash, etc., can be seasoned in dry kilns without danger to the material.* Steaming the lumber is commonly resorted to in order to prevent checking and "casehardening," but not, as has frequently been asserted, to enable the board to dry. Yard-dried lumber is not dry, and its moisture is too un-

* The dry kiln shown in Fig. 99 is operated at this temperature, and live steam is admitted once or twice a day to prevent checking.—J. B. J.

evenly distributed to insure good behavior after manufacture. Careful piling of the lumber both in the yard and kiln, is essential to good drying.

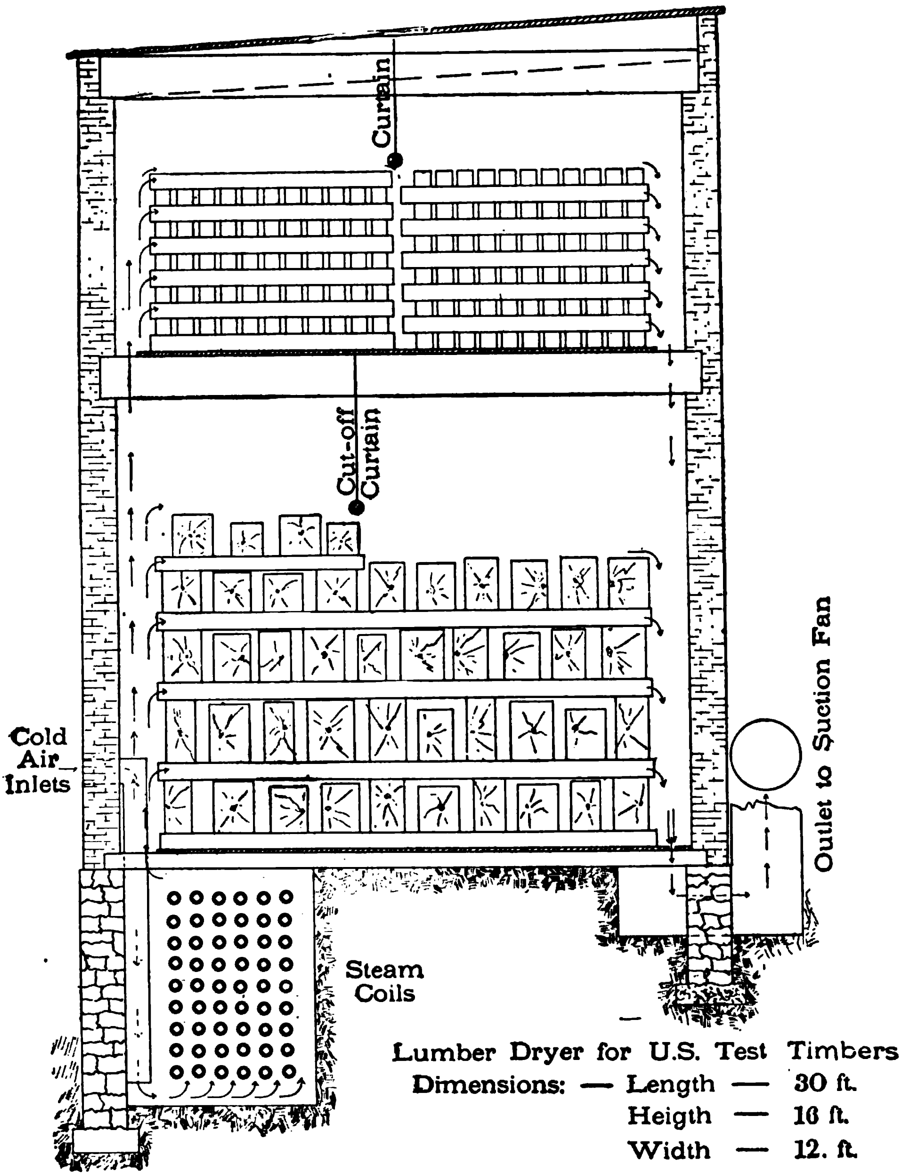

Fig. 99.

Piling boards on edge or standing them on end is believed to hasten drying. This is true only because in either case the air can circulate more freely around them than when they are piled in the ordinary way. Boards on

end dry unequally; the upper half dries much faster than the lower half and horizontal piling is, therefore, preferable.

Since the proportion of sap- and heart-wood varies with size, age, species, and individual, the following figures must be regarded as mere approximations:

POUNDS OF WATER LOST IN DRYING 100 POUNDS OF GREEN WOOD IN THE KILN.

Common Names of Species.	Sapwood or Outer Part.	Heartwood or Interior.
(1) Pines, cedars, spruces, and firs	45–65	16–25
(2) Cypress, extremely variable	50–65	18–60
(3) Poplar, cottonwood, basswood	60–65	40–60
(4) Oak, beech, ash, elm, maple, birch, hickory, chestnut, walnut, and sycamore	40–50	30–40

The lighter kinds have the most water in the sapwood; thus sycamore has more than hickory.

SHRINKAGE OF WOOD.

194. Shrinkage Explained.—When a short piece of wood-fibre, such as that shown in Fig. 100, *A*, is dried it shrinks, its wall grows thinner (as indicated by dotted lines), its width, *ab*, the thickness of the fibre, becomes smaller, and the cavity or opening larger, but, strange to say, the height or length, *bc*, remains the same. In a similar piece of fibre with a thinner wall (Fig. 100, *B*) the effect is the same, but the wall being only half as thick the total change is only about half as great.*

If sections or pieces of fibres are dried and then placed on moist blotting-paper, they will take up water and swell to their original size, though the water has been taken up only by their walls and none has entered into their openings or lumina. This indicates that the water in the cavity or lumen of a fibre has nothing to do with its dimensions, and that if the cell-walls are saturated it makes no difference

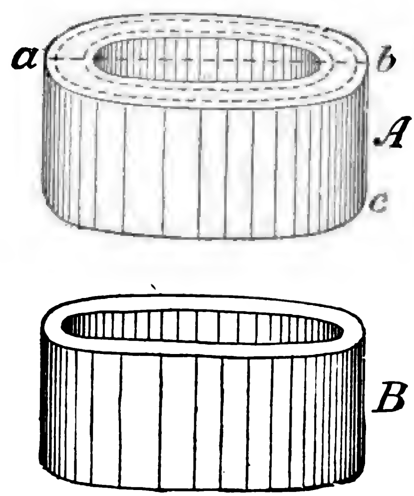

Fig. 100.—Short Pieces of Wood-fibres, one thick-, the other thin-walled; magnified.

* Though generally true, it must not be supposed that the fibres of all species, or even the fibres of the same tree, shrink exactly in proportion to the thickness of their walls.

in the volume of a block of pine wood whether the cell-cavities are empty as in the heartwood or three fourths filled as in the sapwood.

If an entire fibre, as shown in Fig. 101, is dried, the wall at its ends *a* and *b*, like those of the sides, grows thinner, and thereby the length of the entire cell grows shorter. Since this length is often a hundred or more times as great as the diameter, the effect of this shrinkage is inappreciable; and if a long board shrinks lengthwise, it is largely due, as we shall see, to quite another cause.

A thin cross-section of several fibres (see Fig. 102, *A*) like the piece of a single fibre shrinks when dried, the wall of each fibre becomes

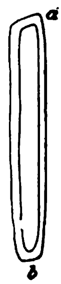

FIG. 101.
Isolated Cell.

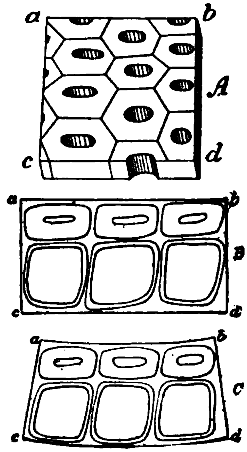

FIG. 102.—Warping of Wood.

thinner, and thus each piece smaller, and the piece on the whole necessarily shares this diminution of size, the distances *ab* and *cd* each becoming shorter. Where the cells are very similar in size and in the thickness of their walls, as in the case of piece *A*, Fig. 102, *ab* and *cd* become shorter by about the same amount; but if the piece is made up of fibres some of which have thin and others thick walls, as piece *B*, Fig. 102, then, the row of thick-walled cells shrinking much more than the row of thin-walled cells, the piece becomes unevenly shrunk or warped as shown in Fig. 102, *C*. Not only is the piece warped, but the force which led to this warping continues to strain the interior parts of the piece in different directions.

Since in all our woods cells with thick walls and cells with thin walls are more or less intermixed, and especially as the spring wood and summer wood nearly always differ from each other in this respect, strains and tendencies to warp are always active when wood dries out, because the summer wood shrinks more than the spring wood, heavier wood in general more than light wood of the same kind.

If the piece *A*, Fig. 102, after drying is placed edgewise on moist blotting-paper, the cells on the under side, at *cd*, take up moisture from the paper and swell before the upper cells at *ab* receive any moisture. This causes the under side of the piece to become longer than the upper side, and, as in the case of piece *C*, warping occurs. Soon, however, the moisture penetrates to all the cells and the piece straightens out. A thin board behaves exactly like this minute piece, only the process is slower and more easily observed. But while a thin board of pine curves laterally, it remains quite straight lengthwise, since in this direction both shrinkage and swelling are small. A thin disk or cross-section swells, and when moistened on one side warps as readily in one direction as in another. If a green board is exposed to the sun upon one side, warping is produced by removal of water and consequent shrinkage of that side, and the course of the process is simply reversed.

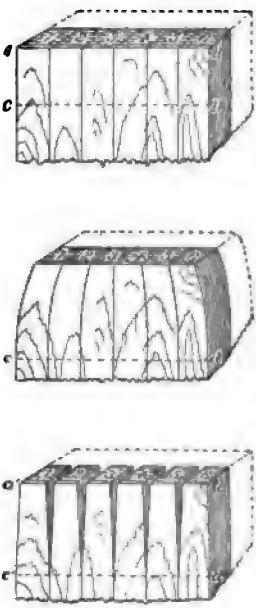

FIG. 103.—Formation of Checks.

As already stated, wood loses water faster from the end than from the longitudinal faces. Hence the ends shrink at a different rate from the interior parts.

195. Effects of Shrinkage.—In a timber, the width *AB* (Fig. 103, *X*) may have shortened (Fig. 21, *Y*), while a short distance from the end *cd* the original width is still preserved. This should produce a bending of the parts toward the centre of the piece as shown in exaggeration at *Y*, but the rigidity of the several parts of the timber prevents such bending and the consequent strain leads to their separation as shown at *Z*, the end surface of the timber being "checked."

As the timber dries out, the line *cd* becomes shorter, the parts 1 to 6 are allowed to approach again, and the checks close up and are no longer visible.

The faster the drying at the surface, the greater is the difference in the moisture of the different parts, and hence the greater the strains and consequently also the amount of checking. This becomes very evident when fresh wood is placed in the sun, and still more in a hot kiln. While most of these smaller checks are thus only temporary, closing up again, some large radial checks remain and even grow larger as drying progresses. Their cause is a different one and will presently be explained.

The temporary checks not only occur at the ends, but are developed on the sides also, only to a much smaller degree. They become especially an-

noying on the surface of thick planks of hard woods, and also on peeled logs when exposed to the sun.

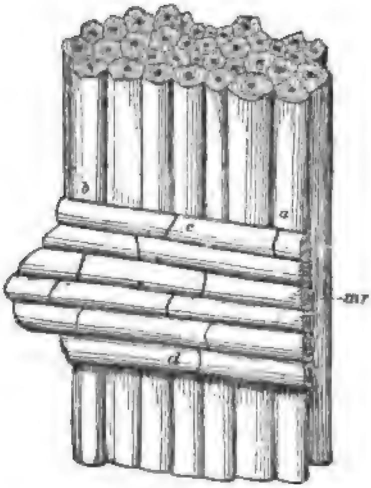

FIG. 104.—Small Pith-ray in Oak. *a*, *b*, wood-fibres; *c*, *d*, cells of pith-ray.

So far we have considered the wood as if made up only of parallel fibres all placed longitudinally in the log. This, however, is not the case. A large part of the wood is formed by the medullary or pith rays. In pine over 15,000 of these occur on a square inch of a tangential section, and even in oak the very large rays, which are readily visible to the eye, represent scarcely a hundredth part of the number which the microscope reveals.

As seen in Fig. 104, the cells of these rays have their length at right angles to the direction of the wood-fibres.

If a large pith-ray of white oak is whittled out and allowed to dry, it is found to shrink greatly in the direction from *c* to *d* (Fig. 104), while, as we have stated, the fibres to which the ray is firmly grown in the wood do not shrink in the same direction. Therefore, in the wood, as the cells of the pith-ray dry, they pull on the longitudinal fibres and try to shorten them, and, being opposed by the rigidity of the fibres, the pith-ray is greatly strained. But this is not the only strain it has to bear. Since the fibers from *a* to *b* (Fig. 104) shrink as much again as the pith-ray in this, its longitudinal direction, the fibres tend to shorten the ray, and the latter, in opposing this, prevents the former from shrinking as much as they otherwise would. Thus the structure is subjected to severe strains at right angles to each other; and herein lies the greatest difficulty of wood-seasoning, for whenever the wood dries rapidly these fibres have not the chance to "give" or accommodate themselves, and hence fibres and pith-rays separate and checks result which, whether visible or not, are detrimental in the use of the wood.

The contraction of the pith-rays parallel to the length of the board is probably one of the causes of the small amount of longitudinal shrinkage which has been observed in boards.* The smaller shrinkage of the pith-rays along the radius of the log (the length of the pith-ray) opposing the shrinkage of the fibres in this direction becomes one of the causes of the second great trouble in wood-seasoning, namely, the difference in the

* In addition to this all fibres having an oblique position, as those at pith-rays and knots, also the oblique, tapering ends of all fibers, contribute to this longitudinal shrinkage, since one component of their normal shrinkage is longitudinal.

amount of the shrinkage along the radius and that along the rings or tangent.

This greater tangential shrinkage appears to be due, in part, to the cause just mentioned, but also to the fact that the greatly shrinking bands of summer wood are interrupted, along the radius, by as many bands of porous spring wood, while they are continuous in the tangential direction. In this direction, therefore, each such band tends to shrink, as if the entire piece were composed of summer wood; and since the summer wood represents the greater part of the wood-substance, this tendency of greater tangential shrinkage prevails.

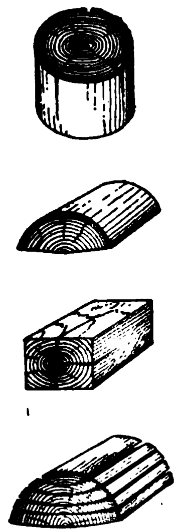

FIG. 105.—Effects of Shrinkage.

The effect of this greater tangential shrinkage affects every phase of woodworking. It leads to permanent checks, and causes the log to split open on drying.

Sawed in two, the flat sides of the log become convex, as in Fig. 105; sawed into a timber, it checks along the median line of the four faces, and if converted into boards the latter take on the forms shown in Fig. 105, all owing to the greater tangential shrinkage of the wood.

Briefly, then, shrinkage of wood is due to the fact that the cell-walls grow thinner on drying. The thicker cell-walls and therefore the heavier wood shrinks most, while the water in the cell-cavities does not influence the volume of the wood. Owing to the great difference of cells in shape, size, and thickness of walls, and still more in their arrangement, shrinkage is not uniform in any kind of wood. This irregular deformation produces stresses which grow with the difference between adjoining cells and are greatest at the pith-rays. These deformations cause warping and checking, and exist even when no outward signs are visible; they are greater if the wood is dried rapidly than if dried slowly, but can never be entirely avoided.

Temporary checks are caused by the more rapid drying of the outer parts of any stick; permanent checks are due to the greater shrinkage, tangentially, along the rings than that along the radius. This, too, is the cause of most of the ordinary phenomena of shrinkage, such as the difference in behavior of entire and quartered logs, "bastard" (tangent) and "rift" (radial) boards, etc., and explains many of the phenomena erroneously attributed to the influence of the bark, or of the greater shrinkage of outer and inner parts of any log.

Once dry, wood may be swelled again to its original size by soaking in water, boiling, or steaming. Soaked pieces, on drying, shrink again as before; boiled and steamed pieces do the same, but to a slightly less degree. Neither hygroscopicity, i.e., the capacity of taking up water, nor shrinkage of wood can be overcome by drying at temperatures below 200° F. Higher

temperatures, however, reduce these qualities, but nothing short of a coaling heat robs wood of the capacity to shrink and swell. Rapidly dried in the kiln, the wood of oak and other hard woods "caseharden," that is, the outer part dries and shrinks before the interior has a chance to do the same, and thus forms a firm shell or case of shrunken, commonly checked wood around the interior. This shell does not prevent the interior from drying, but when this drying occurs, the interior is commonly checked along the medullary rays, as shown in Fig. 106. In practice this occurrence can be prevented by steaming the lumber in the kiln, and still better by drying the wood in the open air or in a shed before placing in the kiln. Since only the first shrinking is apt to check the wood, any kind of lumber which has once been air dried (three to six months for 1-inch stuff) may be subjected to kiln-heat without any danger. Kept in a bent or warped condition during the first shrinking, the wood retains the shape to which it was bent and firmly opposes any attempt at subsequent straightening.

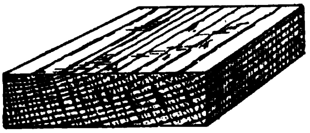

FIG. 106. — "Honeycombed" Board. The checks or cracks form along the pith-rays.

196. Amount of Shrinkage in Timber.—Sapwood, as a rule, shrinks more than heartwood of the same weight, but very heavy heartwood may shrink more than lighter sapwood. The amount of water in wood is no criterion of its shrinkage, since in wet wood most of the water is held in the cavities, where it has no effect on the volume.

The wood of pine, spruce, cypress, etc., with its very regular structure, dries and shrinks evenly and suffers much less in seasoning than the wood of broad-leaved trees. Among the latter, oak is the most difficult to dry without injury. Small-sized split ware and "rift" boards season better than ordinary boards and planks.

To avoid "working" or warping and checking, all high-grade stock is carefully seasoned, preferably in a kiln, before manufacture. Thicker pieces may be made of several parts glued together; larger surfaces are made in panels or of smaller pieces covered with veneer. Boring is sometimes resorted to to prevent the checking of wooden columns.

Since repeated swelling increases the injuries due to seasoning, wood should be protected against moisture when once it is dry.

Since the shrinkage of our woods has never been carefully studied, and since wood, even from the same tree, varies within considerable limits, the figures given in the following table are to be regarded as mere approximations. The shrinkage along the radius and that along the tangent (parallel to the rings) are not stated separately in the following table, and the figures represent an average of the shrinkage in the two directions. Thus, if the shrinkage of soft pine is given at 3 inches per hundred, it means that the sum of radial and tangential shrinkage is about 6 inches, of which about 4 inches fall

to the tangent and 2 inches to the radius, the ratio between these varying from 3 to 2, a ratio which practically prevails in most of our woods.

Since only an insignificant longitudinal shrinkage takes place (being commonly less than 0.1 inch per hundred, though in oak it is much more), the change in volume during drying is about equal to the sum of the radial and tangential shrinkage, or twice the amount of linear shrinkage indicated in the table.

Thus, if the linear average shrinkage of soft pine is 3 inches per hundred, the shrinkage in volume is about 6 cubic inches for each 100 cubic inches of fresh wood, or 6 per cent of the volume.

APPROXIMATE SHRINKAGE OF A BOARD, OR SET OF BOARDS, 100 INCHES WIDE, DRYING IN THE OPEN AIR.

Common Names of Species.	Lateral Shrinkage Inches.
(1) All light conifers (soft pine, spruce, cedar, cypress)	3
(2) Heavy conifers (hard pine, tamarack, yew), honey-locust, box-elder, wood of old oaks	4
(3) Ash, elm, walnut, poplar, maple, beech, sycamore, cherry, black locust	5
(4) Basswood, birch, chestnut, horse chestnut, blue beech, young locust	6
(5) Hickory, young oak, especially red oak	Up to 10

MECHANICAL PROPERTIES OF WOOD.*

197. General View.—Every joist and studding, every rafter, sash, and door, the chair we sit on, the floor we walk on, the wood of the wagon or boat we ride in, are all continually tested as to their stiffness and strength, their hardness and toughness. Every step from the simple splitting of a shingle or stave to the construction of the most elegant carriage or sideboard involves a knowledge not only of one, but of several, of the mechanical properties of the material.

In the shop the fitness of the wood for a given purpose never depends on any one quality alone, but invariably upon a combination of several qualities. A spoke must not only be strong, it must be stiff to hold its shape, it must be tough to avoid shattering to pieces, and it must also be hard or else its tenons will become loose in their mortises.

Selecting wood in this way, the woodworker has learned almost all that is at present known about his material; but in many cases the great difficulty which always attends the judgment of complex phenomena has led to erroneous conclusions, and not a few well-established beliefs have their origin more in accidental errors of observation than in fact.

The experimenter endeavors to avoid this complexity by testing the

* This section of the Bulletin was made very simple for popular comprehension. It seems somewhat out of place, therefore, in a scientific work, but is retained here for the sake of completeness.—J. B. J.

wood for each kind of resistance separately; when tested as to their stiffness, the pieces are all shaped, placed, and loaded alike. The wood is selected with a definite object in view; it is green or dry, clear or knotty, straight or cross-grained, according as he wishes to find out the influence of each of these conditions. If pine and oak are to be compared, the pieces are from the same position in the tree and are tried under exactly the same conditions, and thus the case is simplified.

But even results thus arrived at cannot be used indiscriminately, and the figures on the strength of oak given in any book must not be supposed to apply to all oak if tested in the given manner. This is due to the fact that a piece of wood is not simply a material but a structure, just as much as a railroad-bridge or a balloon frame, and as such varies greatly even in the wood of the same tree, nay, more than that, even in the same year's growth of the same cross-section of a log.

A scantling resists bending; it is stiff. On removal of the load it straightens; it is elastic. A column, a prop, or the spoke of a wagon-wheel resists being crushed endwise. So does the upper side of a joist or beam when loaded, while the under side of the beam or of an axe-handle suffers in tension. The tenons of a window sash or of a door tend to break out their mortises, the wood has to resist shearing along the fibres; the steel edge of the eye tends to cut into the hammer-handle, it tries to shear it across the grain. and every nail, screw, bore-hole, or mortise tends to split the board and tries the wood as to its cleavability, while all "bent" ware, from the wicker basket to the one-piece felly or ship's knee, involves its flexibility.

198. Stiffness.—If 100 pounds placed in the middle of a stick 2 by 2 inches and 4 feet long, supported at both ends, bend or "deflect" this stick one eighth of an inch (in the middle), then 200 pounds will bend it about one-fourth inch, 300 pounds three-eighths inch, the deflection varying directly as the load. Soon, however, a point is reached where an additional 100 pounds adds more than one-eighth inch to the deflection—the limit of elasticity has been reached. Taking another piece from the straight-grained and perfectly clear plank of the same depth and width, but 8 feet long, the load of 100 pounds will cause it to bend not only one-eighth inch, but will deflect it by about 1 inch. Doubling the length reduces the stiffness eightfold. Stiffness then decreases as the cube of the length.

Cutting out a piece 2 by 4 inches and 4 feet long, placing it flatwise so that it is double the width of the former stick, and loading it with 100 pounds, we find it bending only one-sixteenth inch: doubling the width doubles the stiffness.

Setting the same 2 × 4-inch piece on edge, so that it is 2 inches wide and 4 inches deep, the load of 100 pounds bends it only about one sixty-fourth inch: doubling the thickness increases the stiffness about eightfold.

It follows that if we double the length and wish to retain the same stiffness we must also double the thickness of the piece.

A piece of wood is usually stiffer with the annual rings set vertically than if the rings are placed horizontally to the load.

Cross-grained and knotty wood, to be sure, is not as stiff as clear lumber; a knot on the upper side of a joist, which must resist in compression, is, however, not so detrimental as a knot on the lower side, where it is tried in tension.

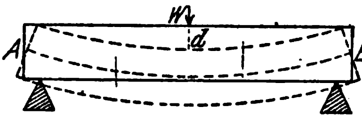

FIG. 107.—Bending a Beam.

Every large timber which comes from the central part of the tree contains knots, and much of its wood is cut more or less obliquely across the grain, both conditions rendering such material comparatively less stiff than small clear pieces.

The same stick of pine green or wet is only about two thirds as stiff as when dry. A heavy piece of long-leaf pine is stiffer than a light piece; heavy pine in general is stiffer than light pine, but a piece of hickory, although heavier than the pine, may not be as stiff as the piece of long-leaf pine; and a good piece of larch exceeds in stiffness any oak of the same weight.

In the same tree stiffness varies with the weight, the heavier wood being the stiffer; thus the heavier wood of the butt log is stiffer than that of the top; timber with much of the heavy summer wood is stiffer than timber of the same kind with less summer wood. In old trees (of pine) the centre of the tree and the sap are the least stiff; in thrifty young pine the centre is the least stiff, but in young second-growth hard woods it is the stiffest.

Since it is desirable, and for many purposes essential, to know beforehand that a given piece with a given load will bend only a given amount, the stiffness of wood is usually stated in a uniform manner and under the term "modulus (measure) of elasticity."

If AB, Fig. 107, is a piece of wood, and d the deflection produced by a weight or load, the stiffness of the wood, as usually stated, is found by the formula

$$\text{Modulus of elasticity} = E = \frac{Wl^3}{4dbh^3},$$

where W is the weight, l the length, b and h the breadth and depth (height) of the stick, and d the deflection for the load W. In the following table the woods are grouped according to their stiffness. The figures are only rough approximations which are based on the data given in Vol. IX of the Tenth Census. The first column contains the above modulus, the second shows how many pounds will produce a deflection of 1 inch in a stick 1 by 1 by 12 inches, assuming that it could endure such bending within the limits of elasticity, and the third column gives the number of pounds which will bend a stick 2 by 2 inches and 10 feet long through 1 inch.

The stick is assumed to rest on both ends; if it is a cantilever, i.e., fastened at one end and loaded at the other, it bears but half as much load at its end for the same deflection.

From the third column it is easy to find how many pounds would bend a piece of the same kind of other dimensions. A 2 × 4-inch bears eight, a 2 × 6-inch twenty-seven times as much as the 2 × 2-inch; a piece 8 feet long is about twice as stiff as a 10-foot piece; a piece 12 feet only about three fifths, 14 feet one third, 16 feet two ninths, 18 feet one sixth, and 20 feet one eighth as stiff.

The number of pounds which will bend any piece of sawed timber by 1 inch may be found by using the formula

$$\text{Necessary weight} = \frac{4Ebh^3}{l^3},$$

where E is the figure in the first column, and b, h, l, the breadth, depth, and length of the timber in inches. If the deflection is not to exceed one-half inch, only one half this load, and if one-fourth inch, only one fourth this load, is permissible; or, in general, $W = \dfrac{4Ebh^3}{dl^3}$, where d is the deflection in inches.

TABLE OF STIFFNESS (MODULUS OF ELASTICITY) OF DRY WOOD.
GENERAL AVERAGES.

Species.	Modulus of Elasticity $E = \frac{Wl^3}{4dbh^3}$ per Square Inch.	Approximate Weight which deflects by Inch a Piece	
		1 by 1 Inch and 12 In. long.	2 by 2 In. and 10 Ft. long.
	Pounds.	Pounds	Pounds.
(1) Live oak, good tamarack, long-leaf, Cuban, and short-leaf pine, good Douglas spruce, Western hemlock, yellow and cherry birch, hard maple, beech, locust, and the best of oak and hickory...........	1,680,000	3,900	62
(2) Birch, common oak, hickory, white and black spruce, loblolly and red pine. cypress, best of ash, elm, and poplar and black walnut......................	1,400,000	3,200	51
(3) Maples, cherry, ash, elm, sycamore, sweet gum, butternut, poplar, basswood, white, sugar, and bull pine, cedars, scrub pine, hemlock, and fir........	1,100,000	2,500	40
(4) Box-elder, horse-chestnut, a number of Western soft pines, inferior grades of hard woods.............	1,000,000	2,300	37

199. Cross-breaking or Bending Strength.—When the addition of 100 pounds to the load on our 2 × 2-inch piece begins to add more than one eighth inch to the deflection, that is, when the stick has been bent beyond its "elastic limit," it still requires an increase of 30 to 50 per cent to the load before the stick breaks. The load which is borne before the limit of elasticity is reached indicates the strength of the wood up to this important point; the load which causes it to break represents its absolute strength, or the "cross-breaking or bending strength" as it is commonly called.

In long-leaf pine the former (modulus of strength at the elastic limit)*

* The "elastic limit" in this case is somewhat of an arbitrary quantity, namely, the point where 100 pounds produces a deflection 50 per cent greater than the first 100 pounds.

is commonly about three fourths of the latter. If left loaded for a consid- time, a load even less than that which brings the stick to its elastic limit will cause it to break, and this load should therefore not be reached in practice.

Unlike the stiffness, the strength of a timber varies approximately with the squares of the thickness and decreases directly with increasing length and not with the cube of this latter dimension. Thus if our piece 2 by 2 inches and 4 feet long can bear 1000 pounds before it breaks, a 2 × 4-inch laid flat will break with about 2000 pounds, and if set edgewise it requires about 4000 pounds to break it, while a piece of the same kind 2 by 2 inches and double the length (8 feet) breaks with half the original load, or only 500 pounds.

All conditions of the material which influence the stiffness also influence the bending strength. Seasoning increases, moisture decreases, the strength; knots and cross-grain depress it, and both are more dangerous on the lower than on the upper side. But while the conifers with their simple cell-structure excel in stiffness, the better hard woods develop the greater strength in bending. Like elasticity and stiffness, the strength is expressed in a uniform manner by the so-called "modulus of rupture," to permit ready estimation of the strength of any given piece. This modulus refers to the resistance per square inch which the parts most strained, "the extreme fibre," offer. The figures usually tabulated are obtained by the formula

$$\text{Strength per square inch of extreme fibre} = f = \frac{3Wl}{2bh^2},$$

where W is the breaking-load, l the length, b and h the breadth and depth of the tested piece of wood.

The following table presents our common woods grouped as to their strength in bending. The load, as before, is supposed to act altogether in the middle. Column 1 gives the strength of the extreme fibre, as explained above; column 2, the number of pounds which will break a piece 1 by 1 inch and 12 inches long; and column 3, the strength of a stick 2 by 2 inches and 10 feet long: from which the strength of any given piece can readily be estimated, allowing, however, for defects, which increase with the size. Thus, if a good piece of pine 2 by 2 inches and 10 feet long breaks with 400 pounds, a 2 × 4-inch set on edge requires 1600 pounds, a 2 × 6-inch, 3600 pounds, a 2 × 8-inch piece 6400 pounds to break it. If a piece 2 by 4 inches and 10 feet long breaks with 1600 pounds, a 2 × 4-inch and 12 feet long piece breaks with about 1300 pounds, one 16 feet with 1000 pounds, etc.; and if a factor of safety of 10 is allowed, only one tenth of the above loads are permissible.

A board ½ inch by 12 inches and 10 feet long contains as much wood as a 3 × 2-inch of the same length, and if placed edgewise should offer four times as much resistance to breaking. Owing to its small breadth,

however, it "twists" when loaded, and in most cases, therefore, bears less than the 2 × 3-inch. To prevent this twisting, joists are braced, and the depth of timbers is made not to exceed four times their thickness.

Short deep pieces shear out or split before their strength in bending can fully be called into play.

To allow for normal irregularities in the structure of wood itself, as well as in the aggregate structure of timbers, an allowance is made on the numbers which have been found by experiment; this allowance is called the "factor of safety." Where the selection of the wood is not very perfect, the load is a variable one, and the safety of human life depends on the structure, the factor is usually taken quite high, as much as 6 or 10; i.e., only one sixth or one tenth of the figures given in the tables is considered safe, and the beam is made six to ten times as heavy as the calculation requires.

STRENGTH IN CROSS-BREAKING OF WELL-SEASONED SELECT PIECES.

Common Names of Species.	Strength of the Extreme Fibre $f = \frac{3Wl}{2bh^2}$ per Square Inch.	Approximate Weight which breaks a Stick: 1 by 1 Inch and 12 Inches long.	Approximate Weight which breaks a Stick: 2 by 2 Inches and 10 Feet long.
	Pounds.	Pounds.	Pounds.
(1) Robinia (locust), hard maple, hickory, oak, birch, best ash and elm, long-leaf, short-leaf, and Cuban pines, tamarack....................	13,000	720	570
(2) Soft maple, cherry, ash, elm, walnut, inferior oak and birch, best poplar, Norway, loblolly, and pitch pines, black and white spruce, hemlock and good cedar......................	10,000	550	440
(3) Tulip, basswood, sycamore, butternut, poplars, white and other soft pines, firs, and cedars...	6,500	350	280

200. Tension and Compression.—When a piece of wood is pulled lengthwise, in the manner shown in Fig. 108, part of the fibres are torn asunder or broken, but many are merely pulled or shredded out from between their neighbors. Since failure in tension thus involves lateral adhesion as well as strength of fibres, it is affected not only by the nature and dimensions of the fibres, but also by their arrangement. Owing to their transverse position the medullary rays (a large part of all woods) offer but one tenth to one twentieth as much resistance as the main body of fibres, and moreover weaken the timber by disturbing the straight course of the fibres and the regularity of the entire structure.

The resistance is also much affected by the position of the grain. The perfectly cross-grained piece *a* (Fig. 109) sustains but about one tenth to one twentieth of the load which is supported by the straight-grained piece *c*, and it is evident that the piece *b*, which represents an excessive degree of cross-grain, is likewise weakened by the oblique position of the grain.

This explains the detrimental influence of a knot on the under side of a board, as in Fig. 110. Since the lower side of the board, in bending, is stretched, the upper side being compressed, the fibres of the lower side are subjected to tension, and the wood of the knot, like the piece of cross-grained

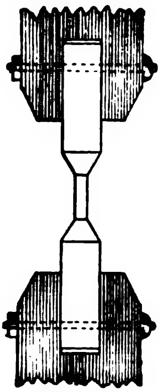

FIG. 108.
Specimen in Tension Test.

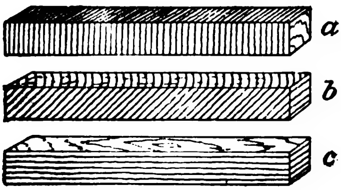

FIG. 109.—Straight and Cross-grained Wood.

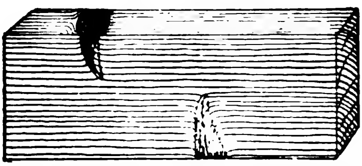

FIG. 110.—Effect of Knots and their Position.

wood, offers but little resistance. Commonly the defect is greatly increased by a season-check in the knot itself, so that the knot affects the strength of the board like a saw-cut of equal depth, but to a less degree.

Tested in compression endwise (Fig. 111), the fibres act as so many hollow columns firmly grown together; and when the load becomes too great the piece fails in the manner illustrated in Fig. 113. This failure is a very complex phenomenon; in wood like pine the fibres of the plane in which failure occurs become separated into small bodies; they tear apart and cease to behave as one solid body, but act as a large number of very small independent pieces. Like the strands of a rope these small bodies offer but little resistance to compression; they bend over, and the piece "buckles."

FIG. 111.—Compression endwise.

It is evident that a vertical position and a regular arrangement of the fibres increase the resistance, and that therefore the medullary rays and oblique position of fibres in cross-grained and knotty timber tend to reduce the strength in compression.

From the following table of strength in tension and compression it will be seen that these two are not always proportional, the stiffer conifers excelling in the latter, the tougher hard woods in the former.

RATIO OF STRENGTH IN TENSION AND COMPRESSION, SHOWING THE DIFFERENCE BETWEEN RIGID CONIFERS AND TOUGH HARD WOODS.

Name of Species.	Ratio: $R = \frac{\text{tensile strength}}{\text{compressive strength}}$.	A Stick 1 Square Inch in Cross-section. Weight required to Pull apart.	Crush endwise.
		Pounds.	Pounds.
Hickory	3.7	32,000	8,500
Elm	3.8	29,000	7,500
Larch	2.3	19,400	8,600
Long-leaf pine	2.2	17,300	7,400

STRENGTH IN COMPRESSION OF COMMON AMERICAN WOODS IN WELL-SEASONED SELECT PIECES.

(Approximate weight per square inch of cross-section requisite to crush a piece of wood endwise.)

	Pounds.
(1) Black locust, yellow and cherry birch, hard maple, best hickory, long-leaf and Cuban pines, and tamarack	9,000+
(2) Common hickory, oak, birch, soft maple, walnut, good elm, best ash, short-leaf and loblolly pines, Western hemlock, and Douglas fir	7,000+
(3) Ash, sycamore, beech, inferior oak, Pacific white cedar, canoe cedar, Lawson's cypress, common red cedar, cypress, Norway and superior spruces, and fir	6,000+
(4) Tulip, basswood, butternut, chestnut, good poplar, white and other common soft pines, hemlock spruce, and fir	5,000+
(5) Soft poplar, white cedar, and some Western soft pines, and firs	4,000+

201. Shearing.—When, in a structure like that shown in Fig. 112, a weight is placed on *J* and the tenon *T* by downward pressure breaks out the piece *ABCD*, this is said to shear out along the fibre. In the same manner, if the shoulder *ABCD* in Fig. 112 is pushed off along *BD*, it is sheared, and if *BD* and *CE* are each 1 inch, the surface thus sheared

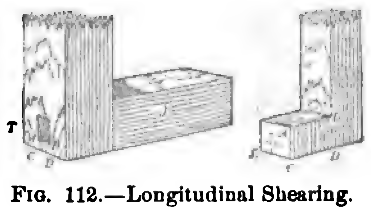

FIG. 112.—Longitudinal Shearing.

off is 1 square inch, and the weight necessary to do this represents the shearing strength per square inch of the particular kind of wood. This resistance is small when compared to that of tension and compression.

Fig. 113.—Various Forms of Failure. *A* and *B*, compression endwise; *C*, shearing (the bolt of a stirrup passed through the mortise and sheared out the end); *D*, tension. The lower figure indicates the number of pounds per square inch which produced the failure in tests by the Division of Forestry. No. 116 (upper figure on each piece) is white pine. Nos. 1, 2, and 5 are long-leaf pine, about one fifth natural size.

In general wet or green wood shears about one third more easily than dry wood; a surface parallel to the rings (tangent) shears more easily than one parallel to the medullary rays. The lighter conifers and hard woods offer less resistance than the heavier kinds, but the best of pine shears one third to one half more readily than oak or hickory, indicating that great shearing strengh is characteristic of "tough" woods.

RESISTANCE TO SHEARING ALONG THE FIBRE.

	Pounds per Square Inch
(1) Locust, oak, hickory, elm, maple, ash, birch	1000 *
(2) Sycamore, long-leaf, Cuban, and short-leaf pine, and tamarack	600
(3) Tulip, basswood, better class of poplar, Norway, loblolly and white pine, spruce, red cedar	400
(4) Softer poplar, hemlock, white cedar, fir	400 †

NOTE.—Resistance to shearing, although a most important quality in wood, has not been satisfactorily studied. The values in the above table, taken from various authors, lack a reliable experimental basis and can be considered as only a little better than guesswork. See Results of Forestry Division Tests in Chapter XXXII.

202. Influence of Weight and Moisture on Strength.—It has been stated that heavy wood is stronger than lighter wood of the same kind, and that seasoning increases all forms of resistance. Let us examine why this is so.

Since the weight of dry wood depends on the number of fibres and the thickness of their walls, there must be more fibres per square inch of cross-section in the heavy than in the light piece of the same kind,‡ and it is but natural that the greater number of fibres should also offer greater resistance, i.e., have the greater strength.

The beneficial influence of drying and consequent shrinking is twofold: (1) In dry wood a greater number of fibres occur per square inch, and (2) the wood-substance itself, i.e., the cell-walls, become firmer. A piece of green long-leaf pine, 1 by 1 inch and 2 inches long, is only about 0.94 by 0.96 inch and 2 inches long when dry; its cross-section is 10 per cent smaller than before, but it still contains the same number of fibres. A dry piece 1 by 1 inch, therefore, contains 10 per cent more fibres than a green piece of the same size, and it is but fair to suppose that its resistance or strength is also about 10 per cent greater.

The influence of the second factor, though unquestionably the more important one, is less readily measured. In 100 cubic inches of wood-substance the material of the cell-walls takes up about 50 cubic inches of water and thereby swells up, becoming about 150 cubic inches in volume. In keeping with this swelling the substance becomes softer and less resistant.

* Over. † Less than.

‡ This imperfect assumption is used only for comparison.

In pine wood this diminution of resistance, according to experiments, seems to be about 50 per cent, and the strength of the substance therefore is inversely as the degree of saturation or solution.

203. Hardness.—Heavy wood is harder than lighter wood; the wood of the butt, therefore, is harder than that of the top, the darker summer wood harder than the light-colored spring wood. Moisture softens, and seasoning, therefore, hardens wood. Wood is much harder when pressed longitudinally than when pressed transversely to the fibres, and it is somewhat stronger tangentially than radially. Though harder wood resists saw and chisel more than softer wood, the working quality of the wood is not always a safe criterion of its hardness.

The following indicates the hardness of our common woods:

1. Very hard woods requiring over 3200 pounds per square inch to produce an indentation of one-twentieth inch: Hickory, hard maple, osage orange, black locust, persimmon, and the best of oak, elm, and hackberry.

2. Hard woods requiring over 2400 pounds per square inch to produce an indentation of one-twentieth inch: Oak, elm, ash, cherry, birch, black walnut, beech, blue beech, mulberry, soft maple, holly, sour gum, honey-locust, coffee-tree, and sycamore.

3. Moderately hard woods, requiring over 1600 pounds per square inch to produce an indentation of one-twentieth inch: The better qualities of Southern and Western hard pine, tamarack and Douglas spruce, sweet gum, and the lighter qualities of birch.

4. Soft woods requiring less than 1600 pounds per square inch to produce an indentation of one-twentieth inch: The greater mass of coniferous woods; pine, spruce, fir, hemlock, cedar, cypress, and redwood; poplar, tulip, basswood, butternut, chestnut, buckeye, and catalpa.

204. Cleavability.—When an axe is struck into a piece of wood as shown in Fig. 114, the cleft projects beyond the blade of the axe and the process is not one of cutting, but of tension across the grain. The axe presses on a lever, *ab*, while the surface in which the transverse tension takes place is reduced almost to a line across the stick at *b*. If the wood is very stiff, the cleft runs far ahead of the axe, the lever-arm *ab* is long, and the resistance to splitting proportionately small. A high modulus of elasticity, therefore, helps splitting, while great shearing strength, a good measure for transverse tension and hardness, hinder it.

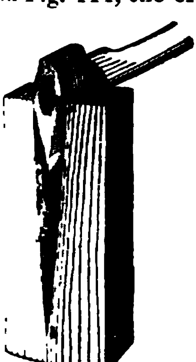

Fig. 114.—Cleavage.

Wood splits naturally along two normal planes, the most readily along the radius, because the arrangement of fibres and pith-rays is radial, and next along the tangent, or with the annual rings, because the softer spring wood forms continuous planes in this direction. Cleavage along the radius, however,

is from 50 to 100 per cent easier, and only in case of cross-grain, etc., the cleavage along the ring becomes the easier. In the wood of conifers, wood-fibres and pith-rays are very regular, the former in perfect radial series or rows, and cleavage is, therefore, very easy in this direction. The same is brought about in the oak by the very wide pith-rays, but where they are thick and narrow, as in sycamore, and generally in the butt cuts and about knots, they impede cleavage by causing a greater irregularity in the course of the wood-fibres. The greater the contrast of spring and summer wood, the easier the cleavage tangentially or in the direction of the rings. This is especially marked in conifers and also in woods like oak, ash, and elm, where the spring wood appears as a continuous series of large pores. Very slow growth influences tangential cleavage, narrow-ringed oak breaks out and splits less regularly even in a radial direction; in conifers, however, this difference scarcely exists. Weight of wood affects the cleavage but little; in heavy wood the entrance of the axe, to be sure, is resisted with more force, but the greater rigidity of the wood, on the other hand, counterbalances this resistance. Irregularities in the course of the fibres, whether spiral growth, cross-grain, or in form of knots, all aid in resisting cleavage. Knotty sticks are split more easily from the upper end, since the cleft then runs around the knots (see Fig. 95). Moisture softens the wood and reduces lateral adhesion, and therefore wood splits more easily when green than when dry.

205. Flexibility.*—Pine is brittle, hickory is flexible; the former breaks, the latter bends. Being the opposite of stiffness, want of stiffness would seem to indicate flexibility. This, however, is only partly true; hickory and ash are stiff and yet among the most flexible of woods. Their small dimensions cause shavings and thin strands of most woods to appear pliable. For this reason the pliable, twisted wicker-willow is not a fair measure of the flexibility of the wood of this species. Generally hard woods are more flexible than conifers, wood of the butt surpassing in this respect that of the main part of the stem, the latter being usually superior to that of the limbs. Moisture softens wood and thereby increases its flexibility. Knots and cross-grain diminish flexibility, but the irregular structure of elm, ash, etc. (particularly the arrangement of bodies of extremely firm fibres, like so many strands, among the softer tissue, as well as the interlacement of fibres due to post-cambial growth), favorably influences the flexibility of these woods.

206. Toughness.†—So far the load by which the exhibition of the various kinds of strength in compression, tension, cross-bending, etc., was pro-

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* The writer here uses "flexibility" as Rankine uses "toughness," that is, the ability to withstand great deformation before rupture. Flexibility, as the opposite of stiffness or rigidity, would signify the readiness to deflect under a given load, which is mathematically shown by a small modulus of elasticity.—J. B. J.

† The writer here uses toughness as indicating resilience, when this term is made to apply to the whole period of deformation and not simply to the elastic field.—J. B. J.

duced has always been assumed as applied slowly and gradually. When a wagon goes lumbering along a cobble pavement, the load on the spokes is not thus applied. Every stone deals the wheel a blow, and a mile's journey means many thousand blows to every wheel-rim and spoke. In chopping, the axe-handle is jarred, and a handle made of pine wood, which shears easily along the fibre, would soon be shattered to pieces. Loads thus applied are "shocks," and resistance to this form of loading requires a combination of various kinds of strength possessed only by "tough" woods. Toughness is a familiar word to woodworkers, and yet is rarely defined. Tough wood must be both strong and pliable. Thus a willow is not tough when dry; it is weak and brittle, and requires, notwithstanding its small lateral dimensions, to be moistened and twisted or sheared into still smaller strands so that its fibres are subjected almost exclusively to tension if great deflection and great strength are to be combined (handles of wicker baskets). Hickory is both strong and pliable ; in the dimensions of a willow twig it can be used almost like a rope. The term "tough," therefore, is properly applied to woods like hickory and elm, and improperly to willow.

Judging from the behavior of elm and hickory, wood may be pronounced "tough" if it offers great resistance to—

(1) Longitudinal shearing over 1000 pounds per square inch,

(2) Tension over 16,000 pounds per square inch,

and permits, when tested dry, of an aggregate combined distortion in compression and tension amounting to not less than 3 per cent.

For instance, of a piece of dry hickory (*H. alba*) we may expect—

Strength in shearing	pounds	1,200
Strength in tension	do.	25,000
Distortion in tension	per cent	2.03
Distortion in compression	do.	1.55
Total distortion	do.	3.58

207. Practical Conclusions.—From the foregoing considerations a few valuable facts, mostly familiar to the thoughtful woodworker, may be deduced

In *framing*, where light and stiff timber is wanted, the conifers excel; where heavy but steady loads are to be supported, the heavier conifers, hard pine, spruce, Douglas spruce, etc., answer as well as hard woods, which are costlier and heavier for the same amount of stiffness. On the other hand, if small dimensions must be used, and especially if moving loads are to be sustained, hard woods are safest, and in all cases where the load is applied in form of "shocks" or jars, only the tougher hard woods should be employed. The heavier wood surpasses the lighter of the same species in all kinds of strength, so that the weight of dry wood and the structural fea-

tures indicative of weight may be used as safe signs in selecting timber for strength.

In *shaping* wood it is better, though more wasteful, to split than to saw, because it insures straight grain and enables a more perfect seasoning.

For *sawed stock* the method of "rift" or "quarter" sawing, which has so rapidly gained favor during the last decade, deserves every encouragement. It permits of better selection and of more advantageous disposition of the wood; rift-sawed lumber is stronger, wears better, seasons well, and is least subject to "working" or warping.

All hardwood material which *checks or warps* badly during seasoning should be reduced to the smallest practicable size before drying, to avoid the injuries involved in this process; and wood once seasoned should never again be exposed to the weather, since all injuries due to seasoning are thereby aggravated. Seasoning increases the strength of wood in every respect, and it is therefore of great importance to protect wooden structures bearing heavy weights against moisture.

Knots, like cross-grain and other defects, reduce the strength of timber. Where choice exists, the knotty side of the joist should be placed uppermost, i.e., should be used in compression.

Season-checks in timber are always a source of weakness; they are more injurious on the vertical than on the horizontal faces of a stringer or joist, and their effect continues even when they have closed up, as many do, and are no longer visible.

Rafted timber, kiln-dried or steamed lumber are, as far as our present knowledge extends, as strong as other kinds; and wherever any of these processes aids in a more uniform or perfect seasoning, it increases the strength of the material.

Pine "bled" for turpentine is as strong as "unbled."

Time of felling, whether season of the year or phase of the moon, does not influence strength, except that summer-felled hard wood rarely seasons as perfectly as that felled in the fall, and to this extent an indirect influence may be observed, as well as by the fact that fungi and insects have a better opportunity for developing.

Warm countries and sunny exposures generally produce heavier and stronger timber, and conditions favorable to the growth of the species also improve its quality. But exceptions occur; neither fast nor slow growth is an infallible sign of strong wood, and it is the character of the annual ring, rather than its width, and particularly the proportion of summer wood, which determines the quality of the material.

CHEMICAL PROPERTIES AND TECHNOLOGICAL PRODUCTS OF WOOD.

208. Chemical Composition.—Wood dried at 300° F. is composed of over 99 per cent of organic and less than 1 per cent of inorganic matter; the latter remains as ashes when wood is burned.

Wood consists of a skeleton of cellulose, permeated by a mixture of other organic substances, collectively designated by the word lignin, and particles of mineral matter or ashes.

Cellulose is the common substance of which plant-cells form their cases or walls; in flax, the entire fibre is almost pure cellulose, but the amount of cellulose obtained from wood, by the common processes, rarely exceeds one half of its dry weight. Cellulose is identical in composition with starch, but unlike the latter it resists alcoholic fermentation, though the plants themselves, as well as decay-producing fungi, are able to reconvert it into starch, from which it seems originally derived, and also to change it into various forms of sugar.* Lignin is as yet a chemical puzzle. The substances forming it are carbohydrates like cellulose itself, but of slightly different proportions and distinguished by greater solubility in acids, and by other chemical properties.

In 100 pounds of wood (dried at 300° F.) and of cellulose the following proportions are found:

	Wood, Pounds.	Cellulose, Pounds.
Carbon	49	44.4
Hydrogen	6	6.1
Oxygen	44	49.3

This composition of wood is fairly uniform for different species.

At ordinary temperatures wood is a very stable compound; both in air and under water it remains the same for centuries, and only when living organisms attack it with their strong solvents and convertants do change and decay set in.

209. Wood as a Fuel.—Heated to 300° F. wood gives off only water, though some slight chemical changes are noticeable even at this temperature. If the heat is increased, gases of pungent odor and taste are evolved; and if the temperature is sufficiently raised, the gases may be ignited, forming the flame of the fire, while the remaining solid part glows like an ignited charcoal, giving much heat, but no flame. The amount of heat produced by wood varies. If first dried at 300° F., 100 pounds of poplar wood should give as much heat as 100 pounds of hickory. In the natural state, however, this is not the case.

The beneficial effect of thorough seasoning for firewood appears from the following consideration:

One hundred pounds of wood as sold in the wood-yards contains in round numbers 25 pounds of water, 74 pounds of wood, and 1 pound of ashes.

The 74 pounds of wood are composed of 37 pounds of carbon, 4.4 pounds of hydrogen, and 32 pounds of oxygen.

* Chemists have succeeded in producing reconversion into grape-sugar; and though the methods thus far employed are expensive, it is to be expected that in the near future wood will become the principal source of both vinegar and alcohol.

In burning (which is a process of oxidation) 4 pounds of hydrogen are already combined with 32 pounds of oxygen, and there are only the 37 pounds of carbon and 0.4 pound of hydrogen available in heat-production. Thus only about one half the weight of the wood-substance itself is heat-producing, while every pound of water combined in the wood requires about 600 units of heat to evaporate it, and thus diminishes the value of the wood as fuel. Hence under the most favorable circumstances 100 pounds of green wood (50 per cent moisture) furnishes about 270,000 units * of heat; 100 pounds of half-dry (30 per cent moisture) about 410,000 units; 100 pounds of air-dry (20 per cent moisture) about 500,000 units; 100 pounds of air-dry (10 per cent moisture) about 580,000 units; 100 pounds of kiln-dry (2 per cent moisture) about 630,000 units.

In the ordinary stove or other small apparatus the evil effect of moisture in the wood is very much increased, since combustion is materially interfered with.

One hundred pounds of ordinary charcoal furnishes 1,200,000 units of heat, but the same quantity of charcoal produced at a temperature of 2000° F. furnishes 140,000 units of heat.

Conifers and the lighter hard woods produce more flame, while the heavy hard woods furnish a good bed of live coal and exceed the former by 25 to 30 per cent in production of heat with ordinary appliances.

210. Charcoal.—Heated in a closed chamber or covered with earth, as in charcoal-pits, the wood is prevented from burning and a variety of changes occur, depending on the rate of heating. If the temperature is raised gradually so that the wood is heated several hours before a temperature of 600° F. is reached, the process is called dry distillation. In this process the wood is destroyed. It forms at first "red" or "brown" coal, still resembling wood, and finally charcoal proper. This coal is darker, heavier, conducts heat and electricity better, requires a greater heat to ignite, and produces more heat per pound in burning the higher the temperature under which it is formed.

One hundred pounds of wood (dried at 300° F.) leaves only about 30 pounds of charcoal. In common practice much less charcoal (18 to 20 per cent) is produced. In this change from wood to coal the volume is diminished by one half, so that a cord of wood which contains about 100 cubic feet of wood solid would be converted into 50 cubic feet at best.

211. Products of Wood-distillation.—Of the 70 pounds of gaseous products which 100 pounds of wood lose, during coaling, in being heated up to 700° F., about 63 pounds become volatile before the temperature of 550° F. is reached.

If condensed in a cooler, about three fourths of the 63 pounds of vol-

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* A unit of heat in this case is the amount of heat which raises the temperature of 1 pound of water 1° F.

atile matter first evolved is found to be wood-vinegar, from which about 4 pounds of pure acetic acid, the only source of perfectly pure vinegar, is obtained. Besides acetic acid, the liquid contains wood-spirits and a quantity of various allied substances.

After the first stage of dry distillation, a large part of the products developed cannot be liquefied in the ordinary cooler. They are gases like the illuminating-gas, mostly belonging to the marsh-gas series; they lack oxygen and thus show that the available oxygen has been nearly exhausted in the preceding part of the process. Products of the latter stages are tars and heavy oils, volatile only at high temperatures. Here also belong the substances, known collectively as wood-creosote, employed as antiseptics in wood-impregnation.

212. Cellulose.—Warmed in dilute nitric acid with a little chlorate of potash, the cells of a piece of wood may be separated. Each cell remains intact, but its wall is reduced in thickness and material; the lignin substances being dissolved out, only the cellulose is left. In commercial-cellulose manufacture, soda, sulphates, and of late chiefly sulphites are substituted for the nitric acid. The wood is chipped, boiled in the respective solution under high pressures, the residue is washed, and the remaining cellulose bleached and ready for use. As a matter of economy the residual liquid is evaporated and the soda used over again.

213. Resin, Turpentine, and Lampblack.—When resinous wood, "fat pine," "lightwood," such as the knots and stumps of long-leaf, pitch, and other pines, is heated in a kiln or retort, the resins ooze out, are collected, and in distillation with steam yield turpentine and resin. The resins and their components vary with the species; the balsam of fir is limpid, its turpentine remains clear on exposure; the resin of pines is very viscid, their turpentines readily oxidize and darken when brought in contact with air. Resins are gathered more commonly either from cracks, such as "wind" and "ring shakes," as in the case of larch and fir (Venetian turpentine), or else from wounds made especially for this purpose, as in the case of naval stores gathered from pines. This latter process is known as "bleeding," "tapping," or "orcharding," and is at present the principal method of obtaining turpentines and resins.

On burning resinous wood, wood-tar, etc., in a smouldering fire, soot is deposited on the walls and partitions of the specially constructed soot-pit. It is then collected, but must be freed of various products of dry distillation, by carefully heating to red heat, before it becomes the lampblack used in printer's ink and otherwise much employed in the arts.

214. Tannin.—Many kinds of wood and the bark of most trees contain tannin. To serve in tanning the bark must contain at least 3 per cent of tannin; the kinds mostly used vary from 5 to 15 per cent, and even the best probably never furnish over 20 per cent in the average. The use of tan-bark involves many disadvantages. It is difficult to dry and preserve, very

liable to mould, it is bulky and therefore expensive to ship and store, and very variable in the amount of tannin which it contains.

To avoid these difficulties the tannic compounds are, in recent times, leached out of the finely ground bark and wood, condensed by evaporation, and shipped as extracts containing 20 to 40 per cent of tannin.

The manufacture of pulp, and the production of fibre capable of being spun and woven, are also technological uses of wood which rely partly upon chemical reactions.

DURABILITY AND DECAY OF WOOD.

215. All Decay Produced by a Fungus-growth.—All wood is equally durable under certain conditions. Kept dry or submerged, it lasts indefinitely. Pieces of pine have been unearthed in Illinois which have lain buried 60 or more feet deep for many centuries. Deposits of sound logs of oak, buried for unknown ages, have been unearthed in Bavaria; parts of the piles of the lake-dwellers, driven more than two thousand years ago, are still intact.

On the radial section of a piece of pine timber, with one of the shelf-like fungus-growths, as shown in Fig. 115, both bark and wood are seen to be affected. A small particle of the half-decayed wood presents pictures like that of Fig. 116. Slender, branching threads are seen to attach themselves closely to the walls of the cells, and to pierce these in all directions. Thus these little threads of fungus mycelium soon form a perfect network in the wood, and as they increase in number they dissolve the walls, and convert the wood-substance and cell-contents into sugar-like food for their own consumption. In some cases it is the woody cell-wall alone that is attacked. In other cases they confine themselves to eating up the starch found in the cells, as shown in Fig. 117, and merely leave a stain (bluing of lumber). In all cases of decay we find the vegetative bodies, these slender threads of fungi, responsible for the mischief. These fine threads are the vegetative body of the fungus; the little shelf is its fruiting-body, on which it produces myriads of little spores (the seeds of fungi). Some fungi attack only conifers, others hard woods; many are confined to one species of tree, and perhaps no one attacks all kinds of wood. One kind produces "red rot," others "bluing." In one case the decayed tracts are tabular, and in the direction of the fibres the wood is "peggy." In other cases no particular shapes are discernible.

Cutting off a disk of loblolly pine, washing it, and then laying it in a clean, shady place in the sawmill, its sapwood will be foumd stained in a few days. Nor is this mischief confined to the surface; it penetrates the sapwood of the entire disk. From this it appears that the spores must have been in the air about the mill, and also that their germination and the growth of the threads or mycelium are exceedingly rapid. (Watching the

progress of mould on a piece of bread teaches the same thing.) Placing a fresh piece of sapwood on ice, another into a dry kiln, and soaking a few others in solutions of corrosive sublimate (mercuric chloride) and other similar salts, we learn that the fungus-growth is retarded by cold, prevented and killed by temperatures over 150° F., and that salts of mercury, etc., have the same effect. The fact that seasoned pieces if exposed are not so

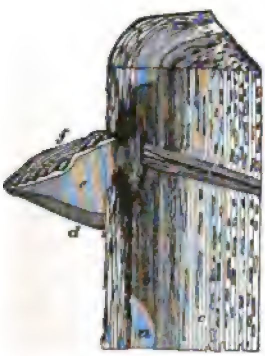

FIG. 115.

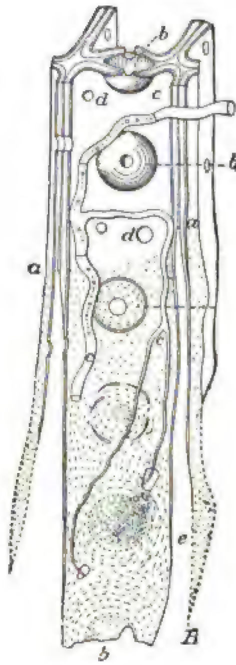

FIG. 116.

FIG. 115.—"Shelf" Fungus on the Stem of a Pine. (Hartig.) *a*, sound wood; *b*, resinous "light" wood; *c*, partly decayed wood or punk; *d*, layer of living spore-tubes; *e*, old filled-up spore-tubes; *f*, fluted upper surface of the fruiting-body of the fungus, which gets its food through a great number of fine threads (the mycelium). its vegetative tissue penetrating the wood and causing its decay.

FIG. 116.—Fungus-threads in Pine Wood. (Hartig.) *a*, cell-wall of the wood-fibres; *b*, bordered pits of these fibres; *c*, thread of mycelium of the fungus; *d*, holes in the cell-walls made by the fungus-threads, which gradually dissolve the walls as shown at *e*, and thus break down the wood-structure.

readily attacked by fungi shows that the moisture in air-dried wood is insufficient for fungus-growth.

From this it appears that warmth, preferably between 60° and 100° F., combined with abundance of moisture (but not immersion), is the most important condition favoring decay, and that the defence lies in the proper regulation or avoidance of these conditions, or else in the use of poisonous salts, which prevent the propagation of fungi.

It is also apparent, therefore, why wood decays faster in Alabama than in Wisconsin, faster in the swamps than on the plains, and why the presence of large quantities of decaying wood about the yard, constantly producing fresh supplies of spores, stimulates decay. Covering with tar or impregnating with creosote, salts of mercury, copper, etc., enables even sapwood to last under the most trying conditions. Contact with the ground assures most favorable moisture conditions for fungus-growth, and the higher temperatures near the surface of the ground, together with the ever-present supply of spores, cause rot in a post to start at the surface more readily than 30 inches below.

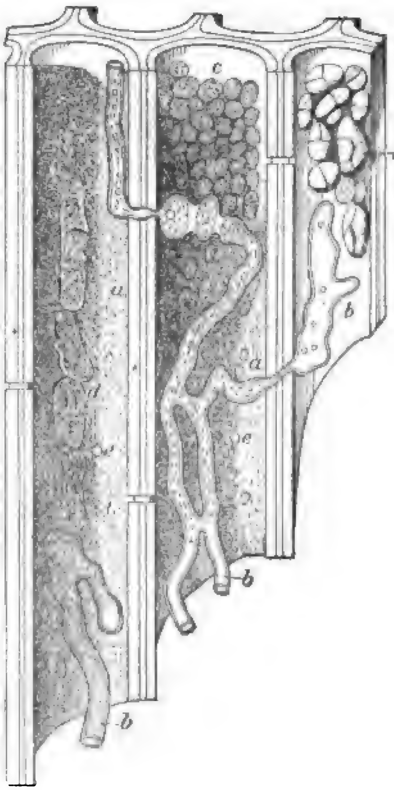

FIG. 117.—Cells of Maple-wood attacked by Fungus-threads (*Nectria cinnabarina* Mayer). Section of three wood-fibres showing the threads of the fungus branching in their cavities and consuming the starch stored in these cells, *a*, interior or cavity of cells; *b*, threads of the fungus; *c*, partly destroyed starch-grains; *d*, dead portions of the fungus-thread together with débris; *e*, holes bored by the fungus through the cell-walls; *S*, starch-grains just being attacked.

216. Prevention of Decay. — The use of means to prevent decay is therefore desirable where timber is placed in positions favorable to fungus-growth, as in railway ties; and all joists and timber in contact with damp brick walls, as also all building material whose perfect seasoning is prevented by the absence of proper circulation of air, should be specially protected. In the former cases it is economy to apply preservative processes; in the latter a sanitary necessity. Wood covered with paint, etc., before it is perfectly seasoned falls a prey to "dry-rot"; the fungus finds abundance of moisture, and the protection intended for the wood protects its enemy, the fungus. Since charcoal resists the solvents of fungi, charring the outer parts of posts makes, if well done, namely, so as not to open checks into the interior of the wood, a very fine protection.

Under ordinary circumstances, only the second great factor of decay, i.e., the moisture condition, can be controlled.

Perfect seasoning, preferably kiln-drying, before using, and protection against the entrance of moisture by tar, paints, and other covers, when put in place, prolong the life of wooden structures. Where such a covering is too expensive, good ventilation at least is necessary. Contact-surfaces,

where timber rests on timber or brick, should in all cases be especially protected.

Different species differ in their resistance to decay. Cedar is more durable than pine, and oak better than beech; but in most cases the conditions of warmth and moisture in particular locations have so much to do with durability that often an oak post outlasts one of cedar, even in the same line of fence, and predictions of durability become mere guesswork.

Containing more ready-made food, and in forms acceptable to a great number of different kinds of fungi, the sapwood is more subject to decay than the heartwood, doubly so where the latter is protected by resinous substances, as in pine and cedar. Several months of immersion improves the durability of sapwood, but only impregnation with preservative salts seems to render it perfectly secure. Once attacked by fungi, wood becomes predisposed to further decay.

Wood cut in the fall is more durable than that cut in summer, only because the low temperature of the winter season prevents the attack of the fungi, and the wood is thus given a fair chance to dry. Usually summer-felled wood, on account of prevalent high temperature and exposure to sun, checks more than winter-felled wood; and since all season-checks favor the entrance of both moisture and fungus, they facilitate destruction. Where summer-felled wood is worked up at once and protected by kiln-drying, no difference exists. (The phases of the moon have no influence whatever on durability!)

In sawing timber much of the wood is bastard-cut; at these places water enters much more readily, and for this reason split and hewn timber and ties generally resist decay perhaps better than if sawed.

The attacks of beetles, as well as those of the shipworm, cannot here be considered; like chisel or saw they are mechanical injuries against which none of our woods are proof, except by impregnation of creosote or other chemical.

RANGE OF DURABILITY IN RAILROAD-TIES.

	Years.		Years.
White oak and chestnut oak	8	Redwood	12
Chestnut	8	Cypress and red cedar	10
Black locust	10	Tamarack	7 to 8
Cherry, black walnut, locust	7	Long-leaf pine	6
Elm	6 to 7	Hemlock	4 to 6
Red and black oaks	4 to 5	Spruce	5
Ash, beech, maple	4		

The durability of wood exposed to the changes of the weather and where painting, after thorough seasoning, is impracticable, is increased by impregnating it with various salts or other chemicals which prevent the fungus from feeding on the wood. The wood is first steamed, to open the pores and remove the hardened surface coating of sap and dirt, and a liquid solution of the preservative material is then injected with the assistance of heat and pressure.

The most efficient fluids used on a large scale are bichloride of zinc and creosote, or both combined. The "life" of railroad-ties is thereby increased to twice and three times its natural duration.

HOW TO DISTINGUISH THE DIFFERENT KINDS OF WOOD.*

217. An Examination of the Structure Essential to Identification.—The carpenter or other artisan who handles different woods becomes familiar with those he employs frequently, and learns to distinguish them through this familiarity, without usually being able to state the characteristic differences. If a wood comes before him with which he is not familiar, he has, of course, no means of determining what it is, and it is possible to select pieces even of those with which he is well acquainted, different in appearance from the average, that will make him doubtful as to their identification. Furthermore, he may distinguish between hard and soft pines, between oak and ash, or between maple and birch, which are characteristically different; but when it comes to distinguishing between the several species of pine or oak or ash or birch, the absence of readily recognizable characteristics is such that but few practitioners can be relied upon to do it. Hence in the markets we find many species mixed and sold indiscriminately.

To identify the different woods it is necessary to have a knowledge of the definite, invariable differences in their structure, besides that of the often variable differences in their appearance. These structural differences may either be readily visible to the naked eye or with a magnifier, or they may require a microsocpical examination. In some cases such an examination cannot be dispensed with if we would make absolutely sure. There are instances, as in the pines, where even our knowledge of the minute anatomical structure is not yet sufficient to make a sure identification.

218. A Structural Key to Species.—In the following key an attempt has been made—the first, so far as we know, in English literature—to give a synoptical view of the distinctive features of the commoner woods of the United States which are found in the markets or are used in the arts. It will be observed that the distinction has been carried in most instances no further than to genera or classes of woods, since the distinction of species can hardly be accomplished without elaborate microscopic study, and also that, as far as possible, reliance has been placed only on such characteristics as can be distinguished with the naked eye or a simple magnifying-glass, in order to make the key useful to the largest number. Recourse has also been taken for the same reason to the less reliable and more variable general external appearance, color, taste, smell, weight, etc.

The user of the key must, however, realize that external appearance, such, for example, as color, is not only very variable, but also very difficult to describe, individual observers differing especially in seeing and describing

* The matter in the remainder of this chapter is mostly the joint product of Dr. B. E. Fernow and Mr. Filibert Roth.

shades of color. The same is true of statements of sizé when relative and not accurately measured, while weight and hardness can perhaps be more readily approximated. Whether any feature is distinctly or only indistinctly seen will also depend somewhat on individual eyesight, opinion, or practice. In some cases the resemblance of different species is so close that only one other expedient will make distinction possible, namely, a knowledge of the region from which the wood has come. We know, for instance, that no long-leaf pine grows in Missouri or Arkansas, and that no white pine can come from Alabama, and we can separate the white cedar, giant arbor vitæ of the West and the arbor vitæ of the Northeast only by the difference of the locality from which the specimen comes. With all these limitations properly appreciated, the key will be found helpful toward greater familiarity with the woods which are more commonly met with.

219. Characteristic Structural Features.—The features which have been utilized in the key and with which (their names as well as their appearance), therefore, the reader must familiarize himself before attempting to use the key, are mostly described as they appear in cross-section. They are:

(1) *Sapwood* and *heartwood* (see Art. 180), the former being the wood from the outer, and the latter from the inner, part of the tree. In some

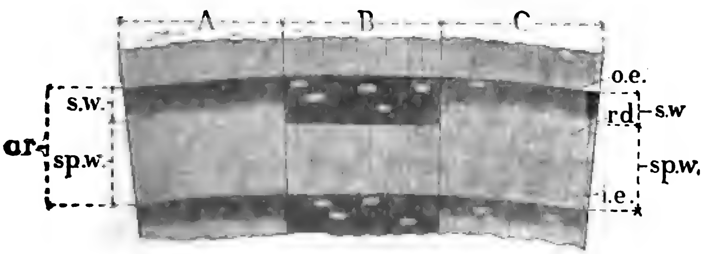

FIG. 118.—"Non-porous" Woods. *A*, fir; *B*, "hard" pine; *C*, soft pine, *ar*, annual ring, *o. e.*, outer edge of ring; *i. e.*, inner edge of ring; *s. w.*, summer wood; *sp. w.*, spring wood; *rd*, resin-ducts.

cases they differ only in shade, and in others in kind of color, the heartwood exhibiting either a darker shade or a pronounced color. Since one cannot always have the two together, or be certain whether he has sapwood or heartwood, reliance upon this feature is, to be sure, unsatisfactory, yet sometimes it is the only general characteristic that can be relied upon. If further assurance is desired, microscopic structure must be examined; in such cases reference has been made to the presence or absence of tracheids in pith-rays and the structure of their walls, especially projections and spirals.

(2) *Annual rings*, their formation having been described in Art. 181. (See also Figs. 118, 120.) They are more or less distinctly marked, and by means of such marking a classification of three great groups of wood is possible.

(3) *Spring wood* and *Summer wood*, the former being the interior (first-formed wood of the year), the latter the exterior (last-formed) part of the

ring. The proportion of each and the manner in which the one merges into the other are sometimes used, but more frequently the manner in which the pores appear distributed in either.

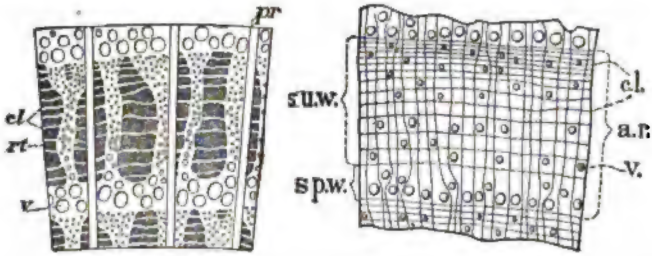

FIG. 119.—"Ring-porous" Woods—White Oak and hickory. *a. r.*, annual ring; *su. w.*, summer wood; *sp. w.*, spring wood; *v*, vessels or pores; *c. l.*, "concentric" lines; *rt*, darker tracts of hard fibres forming the firm part of oak wood; *pr*, pith-rays.

(4) *Pores*, which are vessels cut through, appearing as holes in cross-section, in longitudinal section as channels, scratches, or indentations. (See p. 213 and Figs. 119 and 120.) They appear only in the broad-leaved, so-called, hard woods; their relative size (large, medium, small, minute, and indistinct, when they cease to be visible individually by the naked eye) and manner of distribution in the ring being of much importance, and especially in the summer wood, where they appear singly, in groups, or short broken lines, in continuous concentric, often wavy, lines, or in radial branching lines.

(5) *Resin-ducts* (see p. 210 and Fig. 118), which appear very much like pores in cross-section, namely, as holes or lighter or darker colored dots, but much more scattered. They occur only in coniferous woods, and their presence or absence, size, number, and distribution are an important distinction in these woods.

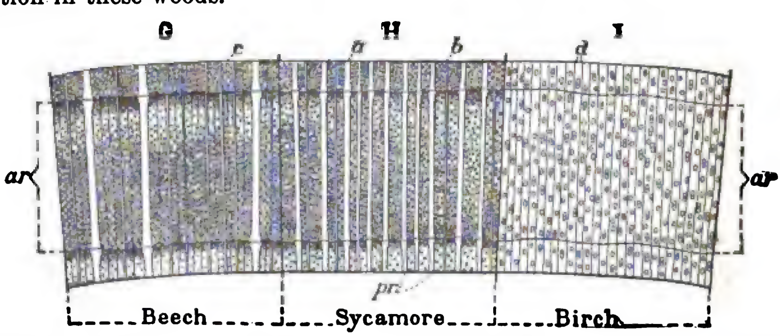

FIG. 120.—"Diffuse-porous" Woods. *ar*, annual ring; *pr*, pith-rays which are "broad" at *a*, "fine" at *b*, "indistinct" at *d*.

(6) *Pith-rays* (see Art. 184 and Figs. 119 and 120), which in cross-section appear as radial lines, and in radial section as interrupted bands of varying breadth, impart a peculiar lustre to that section in some woods.

They are most readily visible with the naked eye or with a magnifier in the broad-leaved woods. In coniferous woods they are usually so fine and closely packed that to the casual observer they do not appear. Their breadth and their greater or less distinctness are used as distinguishing marks, being styled fine, broad, distinct, very distinct, conspicuous, and indistinct when no longer visible by the naked (strong) eye.

(7) *Concentric lines*, appearing in the summer wood of certain species more or less distinct, resembling distantly the lines of pores, but much finer and not consisting of pores. (See Fig. 119.)

Of microscopic features, the following only have been referred to:

(8) *Tracheids*, a description of which is to be found in Art. 185.

(9) *Pits*, simple and bordered, especially the number of simple pits in the cells of the pith-rays, which lead into each of the adjoining tracheids.

For standards of weight, consult table in Art. 191; for standards of hardness, the classification in Art. 203.

Unless otherwise stated the color refers always to the fresh cross-section of a piece of dry wood; sometimes distinct kinds of color, sometimes only shades, and often only general color effects appear.

220. The Use of the Key.—Nobody need expect to be able to use successfully any key for the distinction of woods or of any other class of natural objects without some practice. This is especially true with regard to woods, which are apt to vary much, and when the key is based on such meagre general data as the present. The best course to adopt is to supply one's self with a small sample collection of woods accurately named.* Small, polished tablets are of little use for this purpose. The pieces should be large enough, if possible, to include pith and bark, and of sufficient width to permit ready inspection of the cross-section. By examining these with the aid of the key, beginning with the better-known woods, one will soon learn to see the features described and to form an idea of the relative standards which the maker of the key had in mind. To aid in this, the accompanying illustrations will be of advantage. When the reader becomes familiar with the key, the work of identifying any given piece will be comparatively easy. The material to be examined must, of course, be suitably prepared. It should be moistened; all cuts should be made with a very sharp knife or razor and be clean and smooth, for a bruised surface reveals but little structure. The most useful cut may be made along one of the edges. Instructive, thin, small sections may be made with a sharp penknife or razor, and when placed on a piece of thin glass, moistened and covered with another piece of glass, they may be examined by holding them toward the light.

Finding, on examination with the magnifier, that it contains pores, we know it is not coniferous or nonporous. Finding no pores collected in the spring-wood portion of the annual ring, but all scattered (diffused) through

* Hough's *Wood Sections* will be found both helpful and pleasing. About one hundred and fifty species of American woods are now so prepared by Mr. Romeyn Hough, Lowville, N. Y.—J. B. J.

the ring, we turn at once to the class of "diffuse-porous woods." We now note the size and manner in which the pores are distributed through the ring. Finding them very small and neither conspicuously grouped, nor larger nor more abundant in the spring wood, we turn to the third group of this class. We now note the pith-rays, and finding them neither broad nor conspicuous, but difficult to distinguish even with the magnifier, we at once exclude the wood from the first two sections of this group and place it in the third, which is represented by only one kind, cottonwood. Finding the wood very soft, white, and on the longitudinal section with a silky lustre, we are further assured that our determination is correct. We may now turn to the list of woods and obtain further information regarding the occurrence, qualities, and uses of the wood.

Sometimes our progress is not so easy; we may waver in what group or section to place the wood before us. In such cases we may try each of the doubtful roads until we reach a point where we find ourselves entirely wrong and then return and take up another line; or we may anticipate some of the later-mentioned features and, finding them apply to our specimen, gain additional assurance of the direction we ought to travel. Color will often help us to arrive at a speedy decision. In many cases, especially with conifers, which are rather difficult to distinguish, a knowledge of the locality from which the specimen comes is at once decisive. Thus, Northern white cedar, and bald cypress, and the cedar of the Pacific will be identified even without the somewhat indefinite criteria given in the key.

Engineers and architects can, in the case of the two leading kinds of Southern pine (long-leaf, *P. palustris,* and short-leaf, *P. echinata*), usually determine the species by learning with certainty where the lumber was sawed. This is the more easy with large orders as these are filled directly from the mills, and the shipping bills for the particular cars on which it is delivered may be demanded. The two maps shown in Plates V and VI* will serve to identify the species when the locality is known. It will be seen at once that these two species do not often occupy the same territory.

221. Key to the More important Woods of North America.

[The numbers preceding names refer to the List of Woods following the Key.]

I. Non-porous Woods.—Pores not visible or conspicuous on cross-section even with magnifier. Annual rings distinct by denser (dark-colored) bands of summer wood (Fig. 118).

II. Ring-porous Woods.—Pores numerous, usually visible on cross-section without magnifier. Annual rings distinct by a zone of large pores collected in the spring wood, alternating with the denser summer wood (Fig. 119).

III. Diffuse-porous Woods.—Pores numerous, usually not plainly visible on cross-section without magnifier. Annual rings distinct by a fine line of denser summer-wood cells, often quite indistinct; pores scattered through annual ring, no zone of collected pores in spring wood (Fig. 120).

* These maps are reduced from similar ones published by the Forestry Division of the U. S. Agr. Dept. Washington, as Bulletin No. 13.

NOTE.—The above-described three groups are exogenous, i.e., they grow by adding annually wood on their circumference. A fourth group is formed by the endogenous woods, like yuccas and palms, which do not grow by such additions.

I. NON-POROUS WOODS.

Includes all coniferous woods.

A. Resin-ducts wanting.*

1. No distinct heartwood.
   - *a.* Color effect yellowish white; summer wood darker yellowish (under microscope pith-ray without tracheids)............(Nos. 9–13) FIRS.
   - *b.* Color effect reddish (roseate) (under microscope pith-ray with tracheids).............. ....(Nos. 14 and 15) HEMLOCK.
2. Heartwood present, color decidedly different in kind from sapwood.
   - *a.* Heartwood light orange-red; sapwood pale lemon; wood heavy and hard...(No. 38) YEW.
   - *b* Heartwood purplish to brownish red; sapwood yellowish white; wood soft to medium hard light, usually with aromatic odor. (No. 6) RED CEDAR.
   - *c.* Heartwood maroon to terra cotta or deep brownish red; sapwood light orange to dark amber, very soft and light, no odor; pith-rays very distinct, specially pronounced on radial section ...(No. 7) REDWOOD.
3. Heartwood present, color only different in shade from sapwood, dingy-yellowish brown.
   - *a.* Odorless and tasteless...................... (No. 8) BALD CYPRESS.
   - *b.* Wood with mild resinous odor, but tasteless..(Nos. 1–4) WHITE CEDAR.
   - *c.* Wood with strong resinous odor and peppery taste when freshly cut. (No. 5) INCENSE-CEDAR.

ADDITIONAL NOTES FOR DISTINCTIONS IN THE GROUP.

Spruce is hardly distinguishable from fir, except by the existence of the resin-ducts, and microscopically by the presence of tracheids in the medullary rays. Spruce may also be confounded with soft pine, except for the heartwood color of the latter and the larger, more frequent, and more readily visible resin-ducts.

In the lumber-yard hemlock is usually recognized by color and the slivery character of its surface. Western hemlocks partake of this last character to a less degree.

Microscopically the white pine can be distinguished by having usually only one large pit, while spruce shows three to five very small pits in the parenchyma-cells of the pith-ray communicating with the tracheid.

The distinction of the pines is possible only by microscopic examination. The following distinctive features may assist in recognizing, when in the log or lumber-pile, those usually found in the market:

The light, straw color, combined with great lightness and softness, distinguishes the white pines (white pine and sugar-pine) from the hard pines (all others in the market), which may also be recognized by the gradual change of spring wood into summer wood. This change in hard pines is abrupt, making the summer wood appear as a sharply defined and more or less broad band.

The Norway pine, which may be confounded with the short-leaf pine, can be dis-

* To discover the resin-ducts a very smooth surface is necessary, since resin-ducts are frequently seen only with difficulty, appearing on the cross-section as fine whiter or darker spots normally scattered singly, rarely in groups, usually in the summer wood of the annual ring. They are often much more easily seen on radial, and still more so on tangential, sections, appearing there as fine lines or dots of open structure of different color, or as indentations or pin-scratches in a longitudinal direction.

B. Resin-ducts present.

1. No distinct heartwood; color white, resin-ducts very small, not numerous. (Nos. 33–36) SPRUCE.

2. Distinct heartwood present.

a. Resin-ducts numerous, evenly scattered through the ring.

a'. Transition from spring wood to summer wood gradual; annual ring distinguished by a fine line of dense summer-wood cells; color white to yellowish red: wood soft and light. (Nos. 18–21) SOFT PINES.*

b'. Transition from spring wood to summer wood more or less abrupt; broad bands of dark-colored summer wood; color from light to deep orange; wood medium hard and heavy. (Nos. 22–32) HARD PINES.*

b. Resin-ducts not numerous nor evenly distributed.

a'. Color of heartwood orange-reddish, sapwood yellowish (same as hard pine); resin-ducts frequently combined in groups of 8 to 30, forming lines on the cross-section (tracheids with spirals). (No. 37) DOUGLAS SPRUCE.

b'. Color of heartwood light russett-brown; of sapwood yellowish brown; resin-ducts very few, irregularly scattered (tracheids without spirals)........... .... (Nos. 16 and 17) TAMARACK.

II. RING-POROUS WOODS.

[Some of Group D and cedar-elm imperfectly ring-porous.]

A. Pores in the summer wood minute, scattered singly or in groups, or in short broken lines, the course of which is never radial.

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tinguished by being much lighter and softer. It may also, but more rarely, be confounded with heavier white pine, but for the sharper definition of the annual ring, weight, and hardness.

The long-leaf pine is strikingly heavy, hard, and resinous, and usually very regular and narrow-ringed, showing little sapwood, and differing in this respect from the short-leaf pine and loblolly pine, which usually have wider rings and more sapwood, the latter excelling in that respect.

The following convenient and useful classification of pines into four groups, proposed by Dr. H. Mayr, is based on the appearance of the pith-ray as seen in a radial section of the spring wood of any ring:

Section I. Walls of the tracheids of the pith-ray with dentate projections.

a. One to two large, simple pits to each tracheid on the radial walls of the cells of the pith-ray.—Group 1. Represented in this country only by *P. resinosa*.

b. Three to six simple pits to each tracheid, on the walls of the cells of the pith-ray.—Group 2. *P. tæda*, *palustris*, etc., including most of our "hard" and "yellow" pines.

Section II. Walls of tracheids of pith-ray smooth, without dentate projections.

a. One or two large pits to each tracheid on the radial walls of each cell of the pith-ray.—Group 3. *P. strobus*, *lambertiana*, and other true white pines.

b. Three to six small pits on the radial walls of each cell of the pith-ray. Group 4. *P. parryana* and other nut-pines, including also *P. balfouriana*.

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* Soft and hard pines are arbitrary distinctions, and the two are not distinguishable at the common limit.

1. Pith-rays minute, scarcely distinct.
   - *a.* Wood heavy and hard; pores in the summer wood not in clusters.
     - *a'.* Color of radial section not yellow.............(Nos. 39–44) ASH.
     - *b'.* Color of radial section light yellow; by which, together with its hardness and weight, this species is easily recognized. (No. 103) OSAGE ORANGE.
   - *b.* Wood light and soft; pores in the summer wood in clusters of 10 to 30. (No. 56) CATALPA.
2. Pith-rays very fine, yet distinct; pores in summer wood usually single or in short lines; color of heartwood reddish brown; of sapwood yellowish white: peculiar odor on fresh section................(No. 111) SASSAFRAS.
3. Pith-rays fine, but distinct.
   - *a.* Very heavy and hard; heartwood yellowish brown. (No. 77) BLACK LOCUST.
   - *b.* Heavy; medium hard to hard.
     - *a'.* Pores in summer wood very minute, usually in small clusters of 3 to 8; heartwood light orange-brown. (No. 83) RED MULBERRY.
     - *b'.* Pores in summer wood small to minute, usually isolated; heartwood cherry-red........................(No. 61) COFFEE-TREE.
4. Pith-rays fine, but very conspicuous, even without magnifier. Color of heartwood red; of sapwood pale lemon...........(No. 78) HONEY-LOCUST.

ADDITIONAL NOTES FOR DISTINCTIONS IN THE GROUP.

Sassafras and mulberry may be confounded but for the greater weight and hardness and the absence of odor in the mulberry; the radial section of mulberry also shows the pith-rays conspicuously.

Honey-locust, coffee-tree, and black-locust are also very similar in appearance. The honey-locust stands out by the conspicuousness of the pith-rays, especially on radial sections, on account of their height, while the black locust is distinguished by the extremely great weight and hardness, together with its darker brown color.

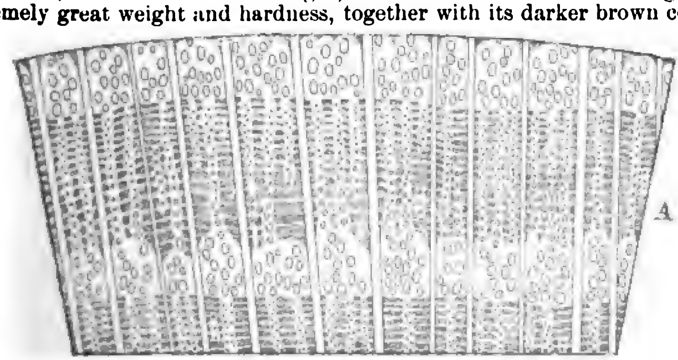

FIG. 121.—Wood of Coffee-tree.

The ashes, elms, hickories, and oaks may, on casual observation, appear to resemble one another on account of the pronounced zone of porous spring wood. The sharply defined large pith-rays of the oak exclude these at once; the wavy lines of pores in the summer wood, appearing as conspicuous finely-feathered hatchings on tangential section, distinguish the elms; while the ashes differ from the hickory by the very conspicuously defined zone of spring-wood pores, which in hickory appear more or less interrupted. The reddish hue of the hickory and the more or less brown hue of the ash may also aid in ready recognition. The smooth, radial surface of split hickory will readily separate it from the rest.

B. Pores of summer wood minute or small, in concentric wavy and sometimes branching lines, appearing as finely-feathered hatchings on tangential section.

1. Pith-rays fine, but very distinct; color greenish white. Heartwood absent or imperfectly developed............................(No. 70) HACKBERRY.
2. Pith-rays indistinct; color of heartwood reddish brown; sapwood grayish to reddish white......................................(Nos. 62–66) ELMS.

C. Pores of summer wood arranged in radial branching lines (when very crowded radial arrangement somewhat obscured).

1. Pith-rays very minute, hardly visible..............(Nos. 58–60) CHESTNUT.
2. Pith-rays very broad and conspicuous..................(Nos. 84–102) OAK.

D. Pores of summer wood mostly but little smaller than those of the spring wood, isolated and scattered; very heavy and hard woods. The pores of the spring wood sometimes form but an imperfect zone. (Some diffuse-porous woods of groups A and B may seem to belong here.)

1. Fine concentric lines (not of pores) as distinct, or nearly so, as the very fine pith-rays; outer summer wood with a tinge of red; heartwood light reddish brown.......................................(Nos. 71–75) HICKORY.
2. Fine concentric lines, much finer than the pith-rays; no reddish tinge in summer wood; sapwood white; heartwood blackish..(No. 105) PERSIMMON.

ADDITIONAL NOTES FOR DISTINCTIONS IN THE GROUP.

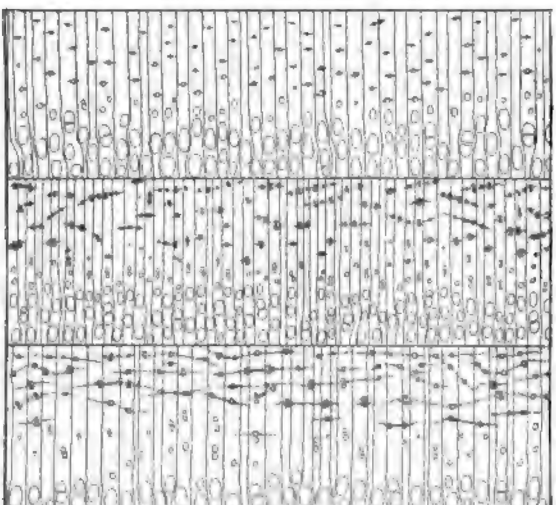

FIG. 122.—*A*, Black Ash; *B*, White Ash; *C*, Green Ash.

The different species of ash may be identified as follows:

1. Pores in the summer wood more or less united into lines.
   a. The lines short and broken, occurring mostly near the limit of the ring. (No. 39) WHITE ASH.
   b. The lines quite long and conspicuous in most parts of the summer wood..................... ....(No. 43) GREEN ASH.
2. Pores in the summer wood not united into lines, or rarely so.
   a. Heartwood reddish brown and very firm...........(No. 40) RED ASH.
   b. Heartwood grayish brown and much more porous..(No. 41) BLACK ASH.

ADDITIONAL NOTES—continued.

In the oaks two groups can be readily distinguished by the manner in which the pores are distributed in the summer wood. In the white oaks the pores are very fine and numerous and crowded in the outer part of the summer wood, while in the black or red oaks the pores are larger, few in number, and mostly isolated. The live oaks, as far as structure is concerned, belong to the black oaks, but are much less porous, and are exceedingly heavy and hard.

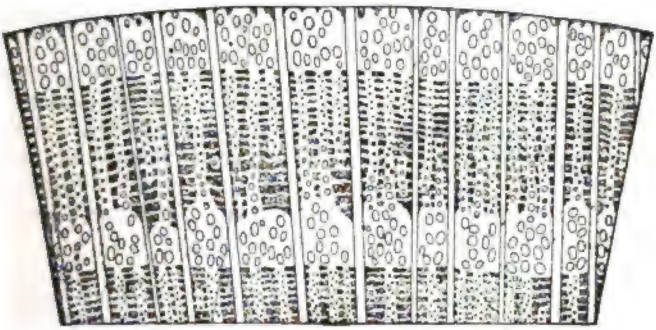

FIG. 123.—Wood of Red Oak. (For White Oak see Fig. 119.)

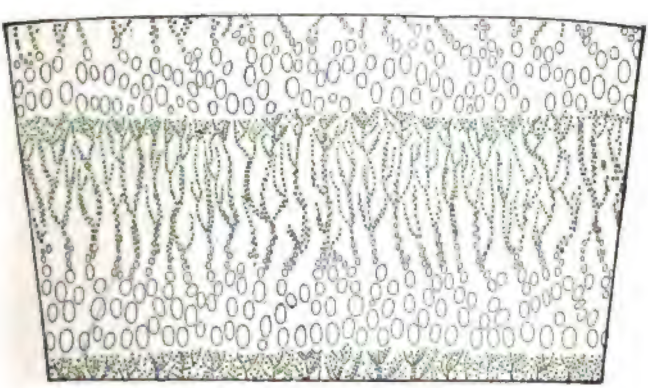

FIG. 124.—Wood of Chestnut.

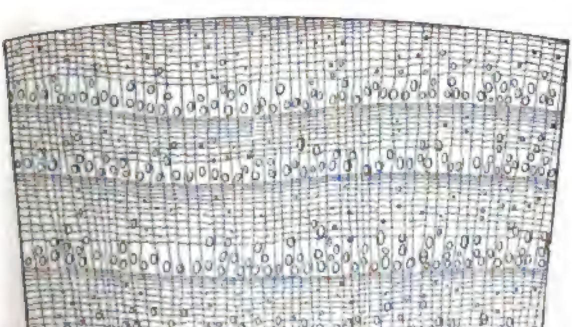

FIG. 125.—Wood of Hickory.

III. DIFFUSE-POROUS WOODS.

[A few indistinctly ring-porous woods of Group II, D, and cedar-elm may seem to belong here.]

A. Pores varying in size from large to minute; largest in spring wood, thereby giving sometimes the appearance of a ring-porous arrangement.

1. Heavy and hard; color of heartwood (especially on longitudinal section) chocolate-brown, ... (No. 116) BLACK WALNUT.
2. Light and soft; color of heartwood light reddish brown..(No. 55) BUTTERNUT.

B. Pores all minute and indistinct; most numerous in spring wood, giving rise to a lighter-colored zone or line (especially on longitudinal section), thereby appearing sometimes ring-porous; wood hard, heartwood vinous-reddish; pith-rays very fine, but very distinct. (See also the sometimes indistinct ring-porous cedar-elm, and occasionally winged elm, which are readily distinguished by the concentric wavy lines of pores in the summer wood.) (No. 57) CHERRY.

C. Pores minute or indistinct, neither conspicuously larger nor more numerous in the spring wood and evenly distributed.

1. Broad pith-rays present.
   a. All or most pith-rays broad, numerous, and crowded, especially on tangential sections, medium heavy and hard, difficult to split. (Nos. 112 and 113) SYCAMORE.
   b. Only part of the pith-rays broad.
      a'. Broad pith-rays well defined, quite numerous; wood reddish white to reddish .... (No. 47) BEECH.
      b'. Broad pith-rays not sharply defined, made up of many small rays, not numerous. Stem furrowed, and therefore the periphery of section, and with it the annual rings, sinuous, bending in and out, and the large pith-rays generally limited to the furrows or concave portions. Wood white, not reddish. (No. 52) BLUE BEECH.

DDITIONAL NOTES FOR DISTINCTIONS IN THE GROUP.

Cherry and birch are sometimes confounded. The high pith-rays on the cherry on radial sections readily distinguish it; distinct pores on birch and spring-wood zone in cherry, as well as the darker vinous-brown color of the latter, will prove helpful.

Two groups of birches can be readily distinguished, though specific distinction is not always possible.

1. Pith-rays fairly distinct, the pores rather few and not more abundant in the spring wood; wood heavy, usually darker. (No. 48) CHERRY-BIRCH and (No. 49) YELLOW BIRCH.

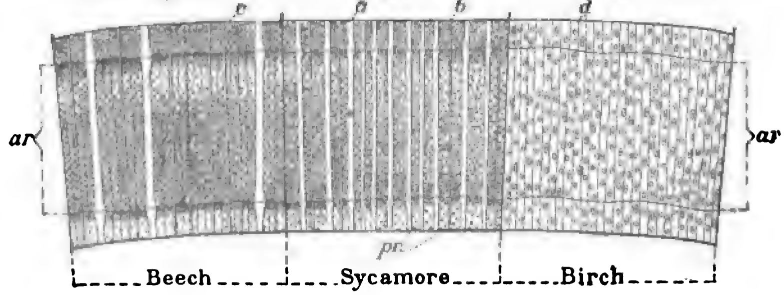

FIG. 126.—Wood of Beech, Sycamore, and Birch.

2. No broad pith-rays present.

a. Pith-rays small to very small, but quite distinct.

a′. Wood hard.

a″. Color reddish white, with dark reddish tinge in outer summer wood........................(Nos. 79–82) MAPLE.

b″. Color white, without reddish tinge........(No. 76) HOLLY.

b′. Wood soft to very soft.

a″. Pores crowded, occupying nearly all the space between pith-rays.

a‴. Color yellowish white, often with a greenish tinge in heartwood...........(No. 115) TULIP-POPLAR.
(No. 116) CUCUMBER-TREE.

b‴. Color of sapwood grayish, of heartwood light to dark reddish brown........(No. 69) SWEET GUM.

b″. Pores not crowded, occupying not over one third the pith-rays; heartwood brownish white to very light brown.
(Nos. 45 and 46) BASSWOOD.

b. Pith-rays scarcely distinct, yet if viewed with ordinary magnifier plainly visible.

a′. Pores indistinct to the naked eye.

a″. Color uniform pale yellow; pith-rays not conspicuous even on the radial section.........(Nos. 53 and 54) BUCKEYE.

b″. Sapwood yellowish gray, heartwood grayish brown; pith-rays conspicuous on the radial section.
(Nos. 67, 68) SOUR GUM.

b′. Pores scarcely distinct, but mostly visible as grayish specks on the cross-section; sapwood whitish, heartwood reddish.
(Nos. 48–51) BIRCH.

3. Pith-rays not visible or else indistinct, even if viewed with magnifier.

1. Wood very soft, white, or in shades of brown, usually with a silky lustre.
(Nos. 105–110) COTTONWOOD (POPLAR).

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2. Pith-rays barely distinct, pores more numerous and commonly forming a more porous spring-wood zone; wood of medium weight.
(No. 51) CANOE- OR PAPER-BIRCH.

The species of maple may be distinguished as follows:

1. Most of the pith-rays broader than the pores and very conspicuous.
(No. 79) SUGAR MAPLE.

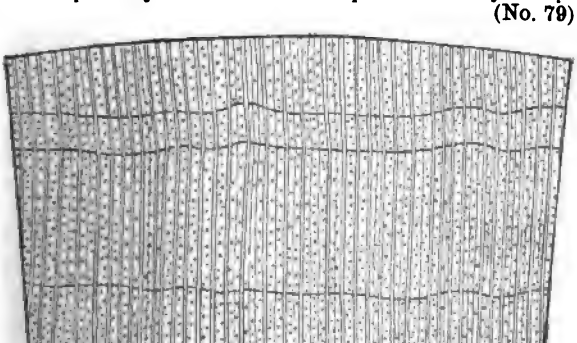

FIG. 127.—Wood of Maple.

ADDITIONAL NOTES—continued.

2. Pith-rays not or rarely broader than the pores, fine but conspicuous.
   a. Wood heavy and hard, usually of darker reddish color and commonly spotted on cross-section......................(No. 80) RED MAPLE.
   b. Wood of medium weight and hardness, usually light-colored. (No. 82) SILVER MAPLE.

Red maple is not always safely distinguished from soft maple. In box-elder the pores are finer and more numerous than in soft maple.

The various species of elm may be distinguished as follows:

1. Pores of spring wood form a broad band of several rows; easy splitting, dark brown heart.....................................(No. 64) RED ELM.
2. Pores of spring wood usually in a single row, or nearly so.
   a. Pores of spring wood large, conspicuously so......(No 62) WHITE ELM.
   b. Pores of spring wood small to minute.
      a'. Lines of pores in summer wood fine, not as wide as the intermediate spaces, giving rise to very compact grain. (No. 63) ROCK-ELM.
      b'. Lines of pores broad, commonly as wide as the intermediate spaces. (No. 66) WINGED ELM.
   c. Pores in spring wood indistinct, and therefore hardly a ring-porous wood. ... .(No. 65) CEDAR-ELM.

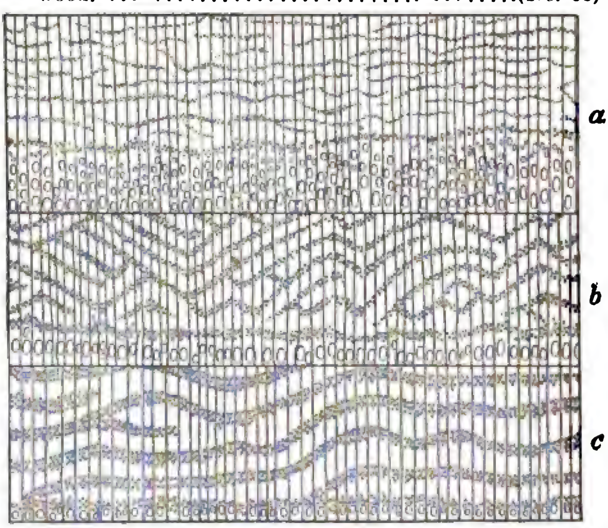

FIG. 128.—Wood of Elm. *a*, Red Elm; *b*, White Elm; *c*, Winged Elm.

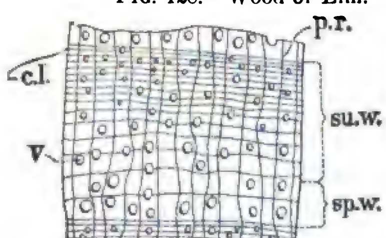

FIG. 129.—Walnut. *p r.*, pith-rays; *c. l.*, concentric lines; *v*, vessels or pores; *su. w.*, summer wood; *sp. w.*, spring wood.

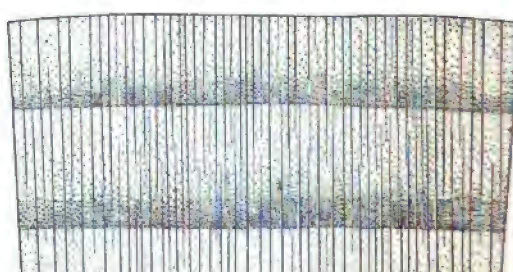

FIG. 130.—Wood of Cherry.

LIST OF THE MORE IMPORTANT WOODS OF THE UNITED STATES.*

[Arranged alphabetically.]

NOTE.—In the following descriptions the terms expressing size have been used with the following meanings:

Small = trees of 50 feet high or less.
Medium = " " 50 to 100 feet high.
Large = " " over 100 feet in height.

All these terms must be understood as having been used as approximate estimates only.

A. CONIFEROUS WOODS.

Woods of simple and uniform structure, generally light, soft but stiff; abundant in suitable dimensions and forming by far the greatest part of all the lumber used.

222. Cedar.—Light, soft, stiff, not strong, of fine texture; sap and heartwood distinct, the former lighter, the latter a dull grayish brown or red. The wood seasons rapidly, shrinks and checks but little, and is very durable. Used like soft pine, but owing to its great durability preferred for shingles, etc. Small sizes used for posts, ties, etc. Cedars usually occur scattered, but they form, in certain localities, forests of considerable extent.

a. **White Cedars.**—Heartwood a light grayish brown.

FIG. 131.—*T. occidentalis.*

1. WHITE CEDAR (*Thuya occidentalis*) (Arborvitæ): Scattered along streams and lakes, frequently covering extensive swamps; rarely large enough for lumber, but commonly used for posts, ties, etc. Maine to Minnesota and northward.

2. CANOE-CEDAR (*Thuya gigantea*) (red cedar of the West): In Oregon and Washington a very large tree, covering extensive swamps; in the mountains much smaller, skirting the watercourses; an important lumber tree. Washington to northern California and eastward to Montana.

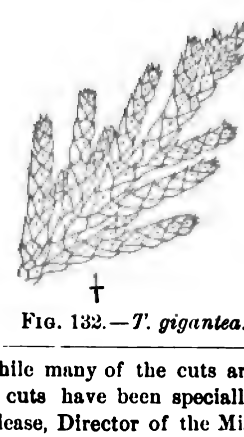

FIG. 132.—*T. gigantea.*

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* The text here is from U. S. Forestry Bulletin No. 10, while many of the cuts are from Apgar's *Trees of the Northern States*. The remaining cuts have been specially drawn for this work, under the direction of Dr. William Trelease, Director of the Missouri Botanical Garden, St. Louis.

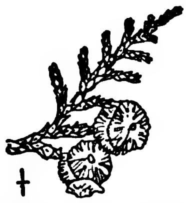


FIG. 133.—*C. thyoides.*

3. WHITE CEDAR (*Chamæcyparis thyoides*): Medium-sized tree, wood very light and soft. Along the coast from Maine to Mississippi.

4. WHITE CEDAR (*Chamæcyparis lawsoniana*) (Port Orford cedar, Oregon cedar, Lawson's cypress, ginger-pine): A very large tree, extensively cut for lumber; heavier and stronger than the preceding. Along the coast-line of Oregon.

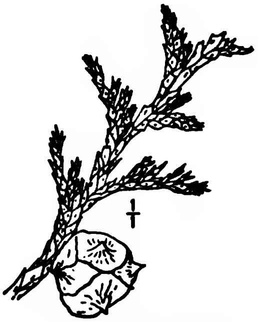


FIG. 134.—*C. lawsoniana.*

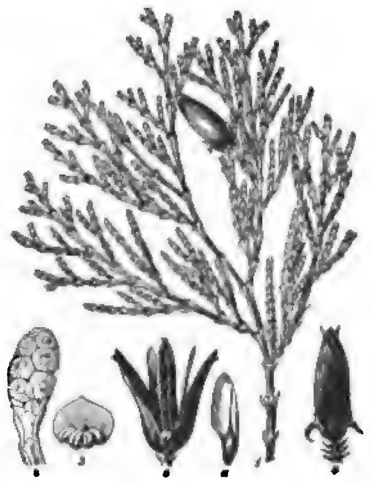


FIG. 135.—*L. decurrens.*

5. WHITE CEDAR (*Libocedrus decurrens*) (incense-cedar): A large tree, abundantly scattered among pine and fir; wood fine-grained. Cascades and Sierra Nevada of Oregon and California.

b. **Red Cedars.**—Heartwood red.

6. RED CEDAR (*Juniperus virginiana*) (Savin juniper): Similar to white cedar, but of somewhat finer texture. Used in cabinet work in cooperage, for veneers, and especially for lead-pencils, for which purpose alone several million feet are cut each year. A small to medium-sized tree scattered through the forests, or, in the West, sparsely covering extensive areas (cedar-brakes). The red cedar is the most widely distributed conifer of the United States, occurring from the Atlantic to the Pacific and from Florida to Minnesota, but attains a suitable size for lumber only in the Southern, and more especially the Gulf, States.

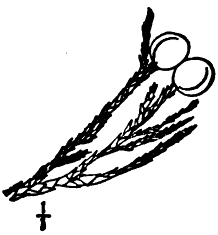


FIG. 136.—*J. virginiana.*

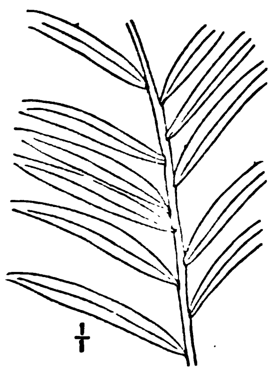


FIG. 137.—*S. sempervirens.*

7. REDWOOD (*Sequoia sempervirens*): Wood in its quality and uses like white cedar; the narrow sapwood whitish; the heartwood light red, soon turning to brownish red when exposed. A very large tree, limited to the coast ranges of California, and forming considerable forests, which are rapidly being converted into lumber.

223. Cypress.

8. CYPRESS (*Taxodium distichum*) (bald cypress; black, white, and red cypress): Wood in appearance, quality, and uses similar to white cedar. "Black cypress" and "white cypress" are heavy and light forms of the same species. The cypress is a large deciduous tree, occupying much of the swamp and overflow land along the coast and rivers of the Southern States.

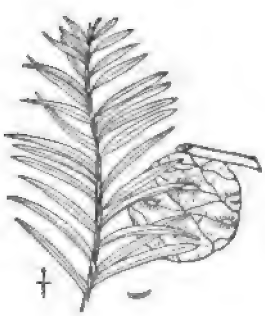
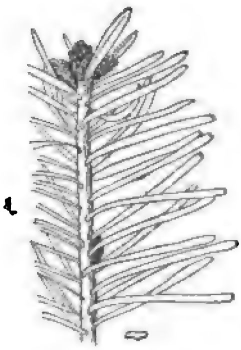


FIG. 138.—*T. distichum.*

224. Fir.—This name is frequently applied to wood and to trees which are not fir; most commonly to spruce, but also, especially in English markets, to pine. It resembles spruce, but is easily distinguished from it, as well as from pine and larch, by the absence of resin-ducts. Quality, uses, and habits similar to spruce.



9. BALSAM-FIR (*Abiesbalsamea*): A medium-sized tree scattered throughout the northern pineries; cut, in lumber operations, whenever of sufficient size, and sold with pine or spruce. Minnesota to Maine and northward.

FIG. 139.—*A. balsamea.*

10. WHITE FIR (*Abies grandis* and *Abies concolor*): Medium to very large-sized tree, forming an important part of most of the Western mountain-forests, and furnishing much of the lumber of the respective regions. The former occurs from Vancouver to central California and eastward to Montana; the latter from Oregon to Arizona and eastward to Colorado and New Mexico.

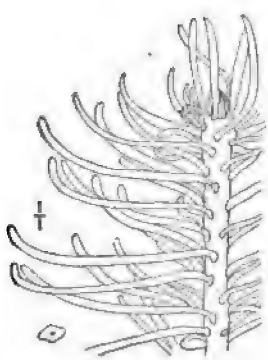


FIG. 140.—*A. grandis.*

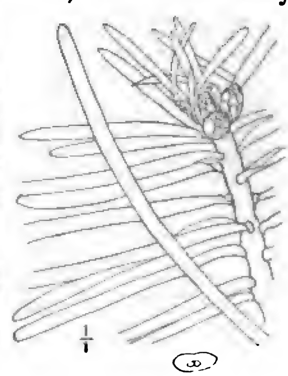


FIG. 141.—*A. concolor.*

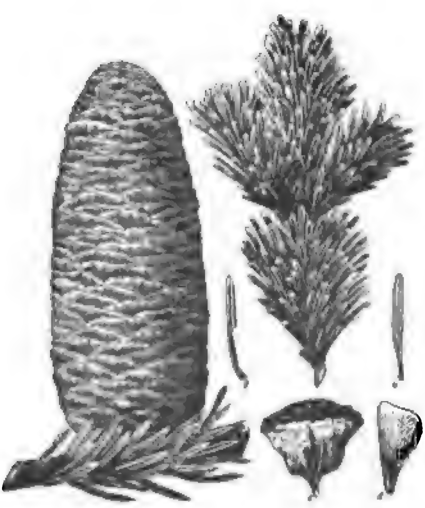


FIG. 142.—*A. amabilis.*

11. WHITE FIR (*Abies amabilis*): Good-sized tree, often forming extensive mountain-forests. Cascade Mountains of Washington and Oregon.

12. RED FIR (*Abies nobilis*) (not to be confounded with Douglas fir; see No. 37): Large to very large tree, forming with *A. amabilis* extensive forests on the slope of the mountains between 3000 and 4000 feet elevation. Cascade Mountains of Oregon.

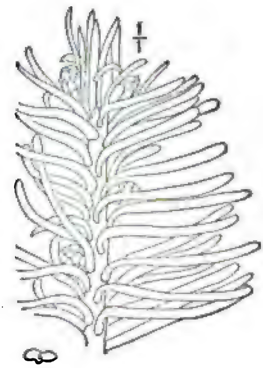


FIG. 143.—*A. nobilis.*

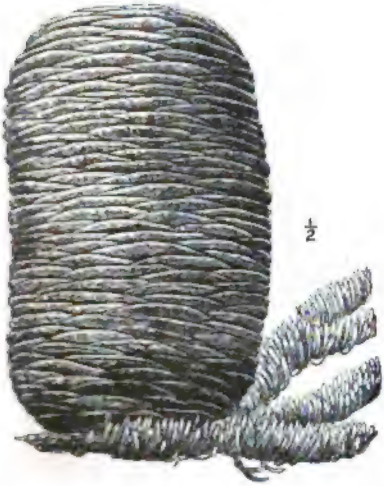


FIG. 144.—*A. magnifica.*

13. RED FIR (*Abies magnifica*): Very large tree, forming forests about the base of Mount Shasta. Sierra Nevada of California, from Mount Shasta southward.

225. Hemlock.—Light to medium weight, soft, stiff but brittle, commonly cross-grained, rough and splintery; sapwood and heartwood not well defined; the wood of a light, reddish-gray color, free from resin-ducts, moderately durable, shrinks and warps considerably, wears rough, retains nails firmly. Used principally for dimension stuff and timbers. Hemlocks are medium to large-sized trees, commonly scattered among broad-leaved trees and conifers, but often forming forests of almost pure growth.

14. HEMLOCK (*Tsuga canadensis*): Medium-sized tree, furnishes almost all the hemlock of the Eastern market. Maine to Wisconsin; also following the Alleghanies southward to Georgia and Alabama.

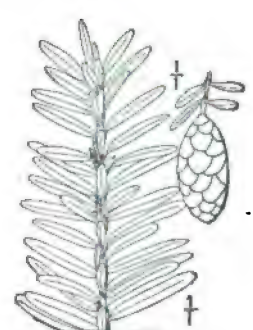


FIG. 145.—*T. canaden-*

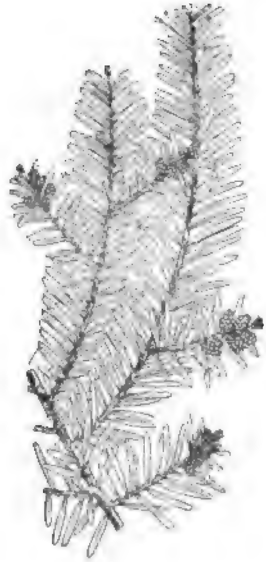


FIG. 146.—*T. mertensiana.*

15. HEMLOCK (*Tsuga mertensiana*): Large-sized tree; wood claimed to be heavier and harder than the Eastern form and of superior quality. Washington to California and eastward to Montana.

226. Larch or Tamarack.—Wood like the best of hard pine both in appearance, quality, and uses, and, owing to its great durability, somewhat preferred in ship-building, for telegraph-poles and railroad-ties. In its structure it resembles spruce. The larches are deciduous trees, occasionally covering considerable areas, but usually scattered among other conifers.

16. TAMARACK (*Larix americana*) (Hackmatack): Medium-sized tree, often covering swamps, in which case it is smaller and of poor quality. Maine to Minnesota, and southward to Pennsylvania.

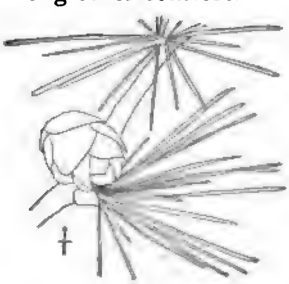


FIG. 147.—*L. americana.*

17. TAMARACK (*L. occidentalis*): Large-sized trees, scattered, locally abundant. Washington and Oregon to Montana.

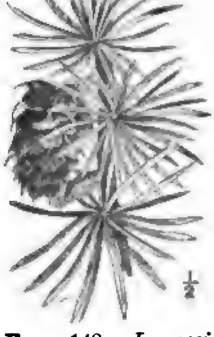


FIG. 148.—*L. occidentalis.*

227. Pine.—Very variable, very light and soft in "soft" pine, such as white pine; of medium weight to heavy and quite hard in "hard" pine, of which long-leaf or Georgia pine is the extreme form. Usually it is stiff, quite strong, of even texture, and more or less resinous. The sapwood is yellowish white; the heartwood, orange-brown. Pine shrinks moderately, seasons rapidly and without much injury; it works easily; is never too hard to nail (unlike oak or hickory); it is mostly quite durable, and if well seasoned is not subject to the attacks of boring-insects. The heavier the wood, the darker, stronger, and harder it is, and the more it shrinks and checks. Pine is used more extensively than any other kind of wood. It is the principal wood in common carpentry, as well as in all heavy construction, bridges, trestles, etc. It is also used in almost every other wood industry, for spars, masts, planks, and timbers in ship-building, in car and wagon construction, in cooperage, for crates and boxes, in furniture work, for toys and patterns, railway-ties, water-pipes, excelsior, etc. Pines are usually large trees with few branches, the straight, cylindrical, useful stem forming by far the greatest part of the tree; they occur gregariously, forming vast forests, a fact which greatly facilitates their exploitation. Of the many special terms applied to pine as lumber, denoting sometimes differences in quality, the following deserve attention:

"White pine," "pumpkin-pine," "soft pine," in the Eastern markets refer to the wood of the white pine (*Pinus strobus*), and on the Pacific Coast to that of the sugar-pine (*Pinus lambertiana*).

"Yellow pine" is applied in the trade to all the Southern lumber pines; in the Northeast it is also applied to the pitch-pine (*P. rigida*); in the West it refers mostly to bull-pine (*P. ponderosa*).

"Yellow long-leaf pine," "Georgia pine," are terms which refer to long-leaf pine (*P. palustris*).

"Hard pine" is a common term in carpentry, and applies to everything except white pine.

"Pitch-pine" includes all Southern pines and also the true pitch-pine (*P. rigida*), but is mostly applied, especially in foreign markets, to the wood of the long-leaf pine (*P. palustris*).

For the great variety of confusing local names applied to the Southern pines in their homes, part of which have been adopted in the markets of the Atlantic seaboard, see report of Chief of Division of Forestry for 1891, page 212, etc., and also the list below.

a. Soft Pines.

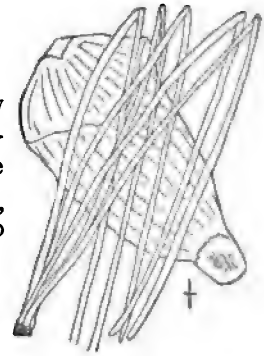


FIG. 149.—*P. strobus.*

18. WHITE PINE (*Pinus strobus*): Large to very large-sized tree; for the last fifty years the most important timber tree of the Union, furnishing the best quality of soft pine. Minnesota, Wisconsin, Michigan, New England, along the Alleghanies to Georgia.

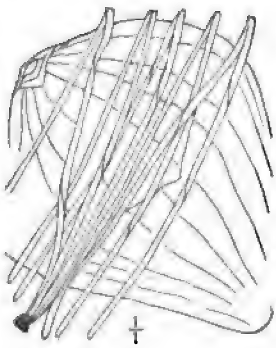


FIG. 150.—*P. lambertiana.*

19. SUGAR-PINE (*Pinus lambertiana*): A very large tree, together with *Abies concolor*, forming extensive forests; important lumber tree. Oregon and California.

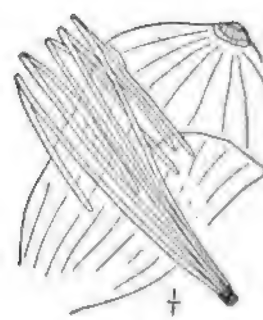


FIG. 151.– *P. monticola.*

20. WHITE PINE (*Pinus monticola*): A large tree, at home in Montana, Idaho, and the Pacific States; most common and locally used in northern Idaho.

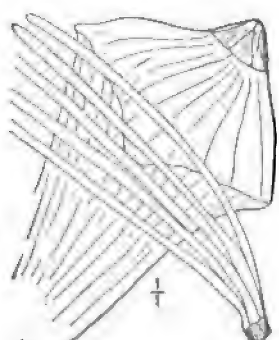


FIG. 152.—*P. flexilis.*

21. WHITE PINE (*Pinus flexilis*): A small tree, forming mountain-forests of considerable extent and locally used; eastern Rocky Mountain slopes; Montana to New Mexico.

b. **Hard Pines.**

22. LONG-LEAF PINE (*Pinus palustris*) (Georgia pine, yellow pine, long straw-pine, etc.): Large tree; forms extensive forests and furnishes the hardest and strongest pine lumber in the market. Coast region from North Carolina to Texas.

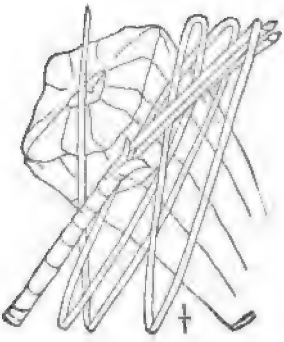


FIG. 153.—*P. palustris.*

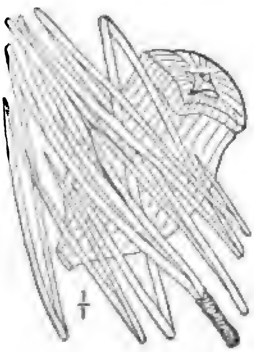


FIG. 154.—*P. ponderosa.*

23. BULL-PINE (*Pinus ponderosa*) (yellow pine): Medium to very large-sized tree, forming extensive forests in Pacific and Rocky Mountain regions; furnishes most of the hard pine of the West; sapwood wide; wood very variable.

24. LOBLOLLY PINE (*Pinus tæda*) (slash-pine, old field-pine, rosemary-pine, sap-pine, short straw-pine, etc.): Large-sized tree, forms extensive forests; wider-ringed, coarser, lighter, softer, with more sapwood than the long-leaf pine, but the two often confounded. This is the common lumber pine from Virginia to South Carolina, and is found extensively in Arkansas and Texas. Southern States; Virginia to Texas and Arkansas.

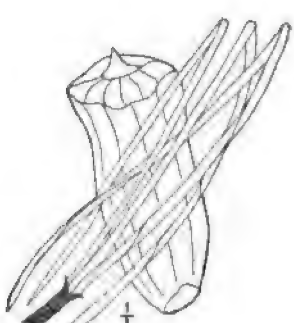


FIG. 155.—*P. tæda.*

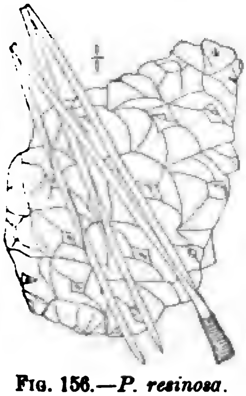


FIG. 156.—*P. resinosa.*

25. NORWAY PINE (*Pinus resinosa*): Large-sized tree, never forming forests, usually scattered or in small groves, together with white pine; largely sapwood and hence not durable. Minnesota to Michigan; also in New England to Pennsylvania.

26. SHORT-LEAF PINE (*Pinus echinata*) (slash-pine, Carolina pine, yellow pine, old field-pine, etc.): Resembles loblolly pine; often approaches in its wood the Norway pine. The common lumber pine of Missouri and Arkansas. North Carolina to Texas and Missouri.

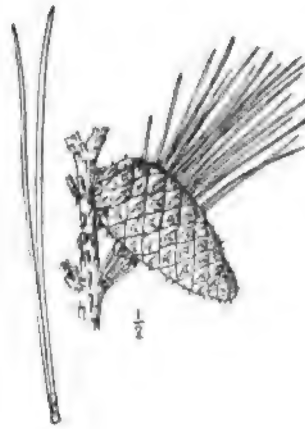


FIG. 157.—*P. echinata.*

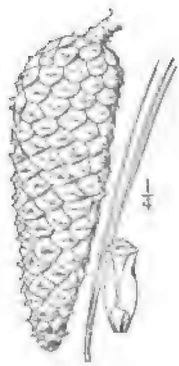


FIG. 158.—*P. cubensis.*

27. CUBAN PINE (*Pinus cubensis*) (slash-pine, swamp-pine, bastard-pine, meadow-pine): Resembles long-leaf pine, but commonly has wider sapwood and coarser grain; does not enter the markets to any great extent. Along the coast from South Carolina to Louisiana.

28. BULL-PINE (*Pinus jeffreyi*) (black pine): Large-sized tree, wood resembling bull-pine (*P. ponderosa*); used locally in California, replacing *P. ponderosa* at high altitudes.

FIG. 159.—*P. jeffreyi.*

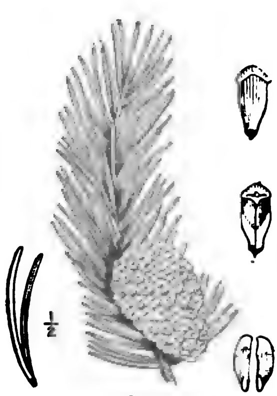


FIG. 160.—*P. murrayana.*

The following are small to medium-sized pines, not commonly offered as lumber in the market; used locally for timber, ties, etc.:

29. BLACK PINE (*Pinus murrayana*) (lodge-pole pine, tamarack): Rocky Mountains and Pacific regions.

30. Pitch-pine (*Pinus rigida*): Along the coast from New York to Georgia, and along the mountains to Kentucky.

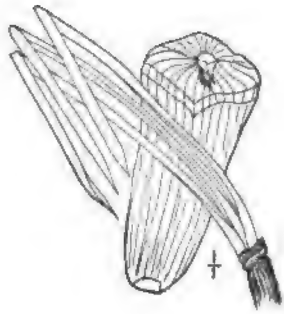


Fig. 161.—*P. rigida.*

31. Jersey Pine (*Pinus inops*) (scrub-pine): As before.

Fig. 162.—*P. inops.*

32. Gray Pine (*Pinus banksiana*) (scrub-pine): Maine, Vermont, and Michigan to Minnesota.

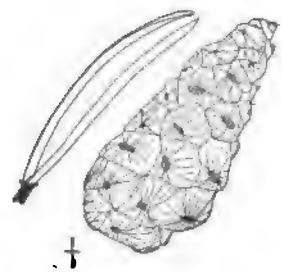


Fig. 163.—*P. banksiana.*

Redwood. See Cedar.

228. Spruce.—Resembles soft pine, is light, very soft, stiff, moderately strong, less resinous than pine; has no distinct heartwood, and is of whitish color. Used like soft pine, but also employed as resonance-wood and preferred for paper pulp. Spruces, like pines, form extensive forests; they are more frugal, thrive on thinner soils, and bear more shade, but usually require a more humid climate. "Black" and "white" spruce, as applied by lumbermen, usually refer to narrow- and wide-ringed forms of the black spruce (*Picea nigra*).

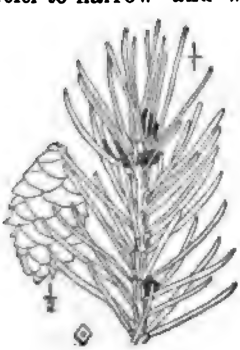


Fig. 164.—*P. nigra.*

33. Black Spruce (*Picea nigra*): Medium-sized tree, forms extensive forests in northeastern United States and in British America; occurs scattered or in groves, especially in low lands throughout the Northern pineries. Important lumber tree in Eastern United States. Maine to Minnesota, British America, and on the Alleghanies to North Carolina.

34. WHITE SPRUCE (*Picea alba*): Generally associated with the preceding; most abundant along streams and lakes, grows largest in Montana, and forms the most important tree of the subarctic forest of British America. Northern United States, from Maine to Minnesota, also from Montana to Pacific, British America.

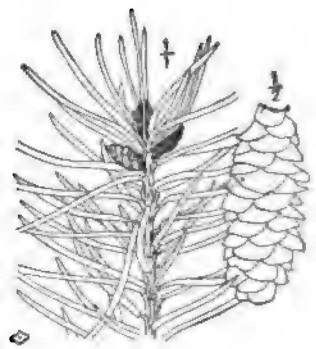


FIG. 165.—*P. alba.*

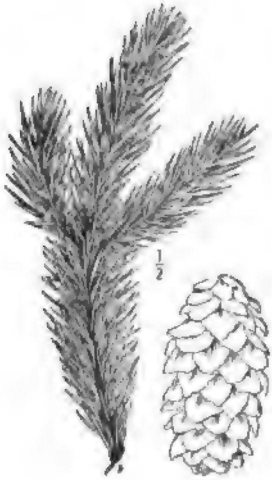


FIG. 166.—*P. engelmanni.*

35. WHITE SPRUCE (*Picea engelmanni*): Medium to large-sized tree, forming extensive forests at elevations from 5000 to 10,000 feet above sea-level; resembles the preceding, but occupies a different station. A very important timber tree in the central and southern parts of the Rocky Mountains. Rocky Mountains from Mexico to Montana.

36. TIDE-LAND SPRUCE (*Picea sitchensis*): A large-sized tree, forming an extensive coast-belt forest. Along the seacoast from Alaska to central California.

Bastard Spruce.—Spruce or fir in name, but resembling hard pine or larch in the appearance, quality, and uses of its wood.

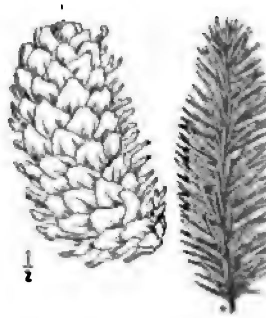


FIG. 167.—*P. sitchensis.*

FIG. 168.—*P. douglasii.*

37. DOUGLAS SPRUCE (*Pseudotsuga douglasii*) (yellow fir, red fir, Oregon pine): One of the most important trees of the Western United States; grows very large in the Pacific States, to fair size in all parts of the mountains, in Colorado up to about 10,000 feet above sea-level; forms extensive forests, often of pure growth. Wood very variable, usually coarse-grained and heavy, with very pronounced summer wood, hard and strong ("red" fir), but often fine-grained and light ("yellow" fir). It replaces hard pine and is especially suited to heavy construction. From the plains to the Pacific Ocean; from Mexico to British America.

Tamarack. See LARCH.

229. Yew.—Wood heavy, hard, extremely stiff and strong, of fine texture, with a pale yellow sapwood and an orange-red heart; seasons well and is quite durable. Yew is extensively used for archery, bows, turner's ware, etc. The yews form no forests, but occur scattered with other conifers.

FIG. 169.—*T. brevifolia.*

38. YEW (*Taxus brevifolia*): A small to medium-sized tree of the Pacific region.

B. BROAD-LEAVED WOODS. (HARDWOODS.)

Woods of complex and very variable structure and therefore differing widely in quality, behavior, and consequently in applicability to the arts.

230. Ash.—Wood heavy, hard, strong, stiff, quite tough, not durable in contact with soil, straight-grained, rough on the split surface and coarse in texture. The wood shrinks moderately, seasons with little injury, stands well and takes a good polish. In carpentry ash is used for finishing lumber, stairways, panels, etc.; it is used in ship-building, in the construction of cars, wagons, carriages, etc., in the manufacture of farm-implements, machinery, and especially of furniture of all kinds, and also for harness work; for barrels, baskets, oars, tool-handles, hoops, clothespins, and toys. The trees of the several species of ash are rapid growers, of small to medium height with stout trunks; they form no forests, but occur scattered in almost all our broad-leaved forests.

39. WHITE ASH (*Fraxinus americana*): Medium, sometimes large-sized tree. Basin of the Ohio, but found from Maine to Minnesota and Texas.

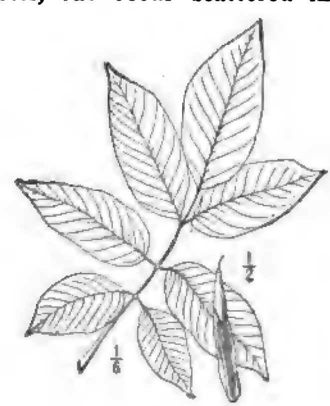


FIG. 170.—*F. americana.*

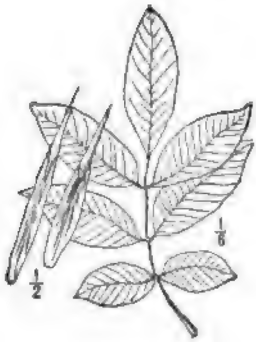


FIG. 171.—*F. pubescens.*

40. RED ASH (*Fraxinus pubescens*): Small-sized tree. North Atlantic States, but extends to the Mississippi.

41. BLACK ASH (*Fraxinus sambucifolia*) (hoop-ash, ground-ash): Medium-sized tree, very common. Maine to Minnesota, and southward to Virginia and Arkansas.

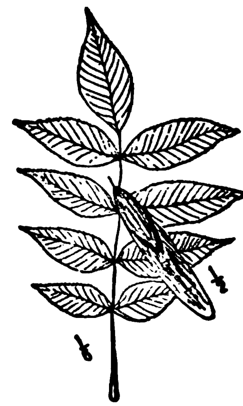


FIG. 172.—*F. sambucifolia.*

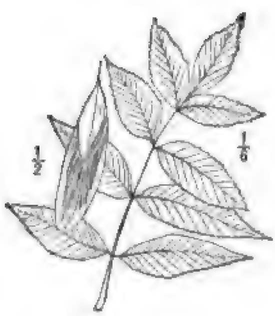


FIG. 173.—*F. quadrangulata.*

42. BLUE ASH (*Fraxinus quadrangulata*): Small to medium-sized. Indiana and Illinois; occurs from Michigan to Minnesota and southward to Alabama.

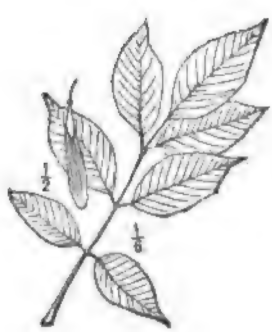


FIG. 174.—*F. viridis.*

43. GREEN ASH (*Fraxinus viridis*): Small-sized tree. New York to the Rocky Mountains, and southward to Florida and Arizona.

44. Oregon Ash (*Fraxinus oregana*): Medium-sized tree. Western Washington to California.

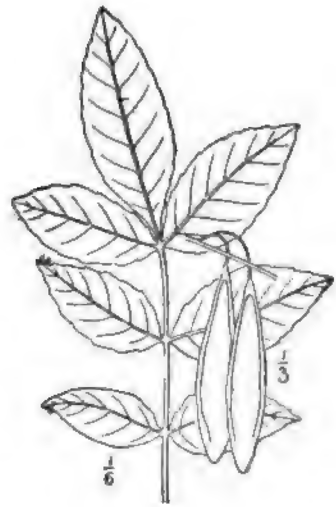


Fig. 175.—*F. oregana.*

Aspen. See Poplar.

231. Basswood.

Fig. 176.—*T. americana.*

45. Basswood (*Tilia americana*) (lime-tree, American linden, lin, bee-tree): Wood light, soft, stiff but not strong, of fine texture, and white to light brown color. The wood shrinks considerably in drying, works and stands well; it is used in carpentry, in the manufacture of furniture and woodenware, both turned and carved, in cooperage, for toys, also for panelling of car and carriage bodies. Medium to large-sized tree, common in all Northern broad-leaved forests; found throughout the Eastern United States.

46. White Basswood (*Tilia heterophylla*): A small-sized tree most abundant in the Alleghany region.

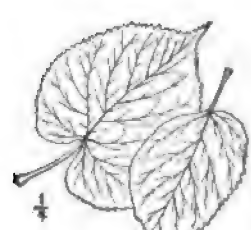


Fig. 177.—*T. heterophylla.*

232. Beech.

47. BEECH (*Fagus ferruginea*): Wood heavy, hard, stiff, strong, of rather coarse texture, white to light brown, not durable in the ground, and subject to the inroads of boring-insects; it shrinks and checks considerably in drying, works and stands well, and takes a good polish. Used for furniture, in turnery, for handles, lasts, etc. Abroad it is very extensively employed by the carpenter, millwright, and wagon-maker, in turnery as well as wood-carving. The beech is a medium-sized tree, common, sometimes forming forests; most abundant in the Ohio and the Mississippi basin, but found from Maine to Wisconsin and southward to Florida.

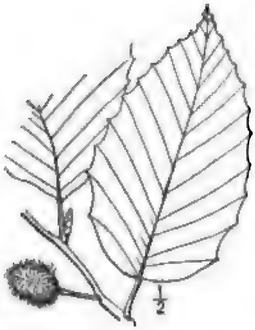


FIG. 178.—*F. ferruginea.*

233. Birch.—Wood heavy, hard, strong, of fine texture; sapwood whitish, heartwood in shades of brown with red and yellow; very handsome, with satiny lustre, equalling cherry. The wood shrinks considerably in drying, works and stands well and takes a good polish, but is not durable if exposed. Birch is used for finishing-lumber in building, in the manufacture of furniture, in wood-turnery for spools, boxes, wooden shoes, etc., for shoe lasts and pegs, for wagon-hubs, ox-yokes, etc., also in wood-carving. The birches are medium-sized trees, form extensive forests northward, and occur scattered in all broad-leaved forests of the Eastern United States.

48. CHERRY-BIRCH (*Betula lenta*) (black birch, sweet birch, mahogany-birch): Medium-sized tree; very common. Maine to Michigan and to Tennessee.

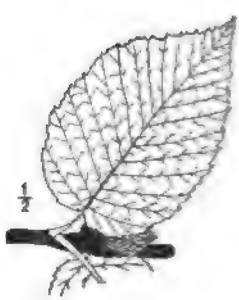


FIG. 179.—*B. lenta.*

49. YELLOW BIRCH (*Betula lutea*) (gray birch): Medium-sized tree; common. Maine to Minnesota and southwest to Tennessee.

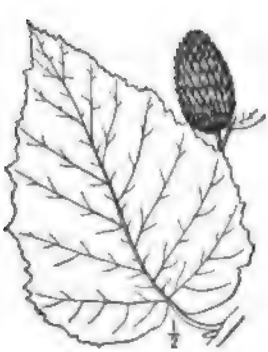


FIG. 180.— *B. lutea.*

50. RED BIRCH (*Betula nigra*) (river-birch): Small to medium-sized tree; very common; lighter and less valuable than the preceding. New England to Texas and Missouri

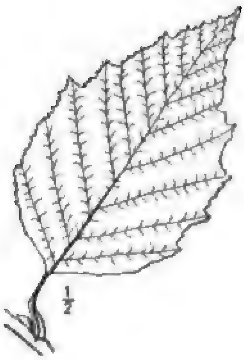


FIG. 181.—*B. nigra.*

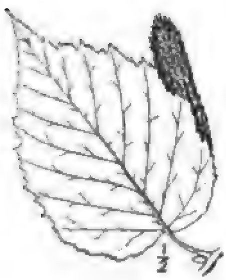


FIG. 182.—*B. papyrifera.*

51. CANOE-BIRCH (*Betula papyrifera*) (white birch, paper-birch): Generally a small tree; common, forming forests; wood of good quality, but relatively light. All along the northern boundary of United States and northward, from the Atlantic to the Pacific.

Black Walnut. See WALNUT.

234. Blue Beech.

52. BLUE BEECH (*Carpinus caroliniana*) (hornbeam, water-beech, ironwood): Wood very heavy, hard, strong, very stiff, of rather fine texture and white color; not durable in the ground; shrinks and checks greatly, but works and stands well. Used chiefly in turnery for tool-handles, etc. Abroad much used by millwrights and wheelwrights. A small tree, largest in the Southwest, but found in nearly all parts of the Eastern United States.

FIG. 183.—*C. caroliniana.*

Bois d'Arc. See OSAGE ORANGE.

235. Buckeye—Horse-Chestnut.—Wood light, soft, not strong, often quite tough, of fine and uniform texture and creamy-white color. It shrinks considerably, but works and stands well. Used for wooden ware, artificial limbs, paper-pulp, and locally also for building-lumber. Small-sized trees, scattered.

FIG 184.
Æ. glabra.

53. OHIO BUCKEYE (*Æsculus glabra*) (fetid buckeye): Alleghanies, Pennsylvania to Indian Territory.

FIG 185.—*Æ. flava.*

54. SWEET BUCKEYE (*Æsculus flava*): Alleghanies, Pennsylvania to Texas.

236. Butternut.

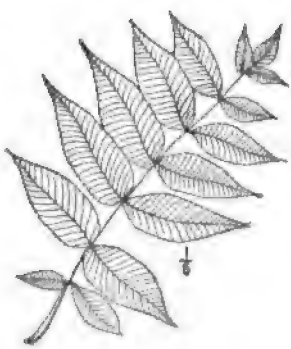


FIG. 186.—*J. cinerea.*

55. BUTTERNUT (*Juglans cinerea*) (white walnut): Wood very similar to black walnut, but light, quite soft, not strong, and of light-brown color. Used chiefly for finishing lumber, cabinetwork, and cooperage. Medium-sized tree, largest and most common in the Ohio basin; Maine to Minnesota and southward to Georgia and Alabama.

237. Catalpa.

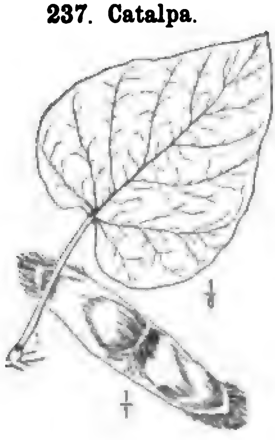


FIG. 187.—*C. speciosa.*

56. CATALPA (*Catalpa speciosa*): Wood light, soft, not strong, brittle, durable, of coarse texture and brown color; used for ties and posts, but well suited for a great variety of uses. Medium-sized trees; lower basin of the Ohio River, locally common. Extensively planted, and therefore promising to become of some importance.

238. Cherry.

57. CHERRY (*Prunus serotina*): Wood heavy, hard, strong, of fine texture; sapwood yellowish white, heartwood reddish to brown. The wood shrinks considerably in drying, works and stands well, takes a good polish, and is much esteemed for its beauty. Cherry is chiefly used as a decorative finishing-lumber for buildings, cars, and boats, also for furniture and in turnery. It is becoming too costly for many purposes for which it is naturally well suited. The lumber-furnishing cherry of this country, the wild black cherry (*Prunus serotina*), is a small to medium-sized tree, scattered through many of the broad-leaved woods of the western slope of the Alleghanies, but found from Michigan to Florida and west to Texas. Other species of this genus as well as the hawthorns (*Cratægas*) and wild apple (*Pyrus*) are not commonly offered in the market. Their wood is of the same character as cherry, often even finer, but in small dimensions.

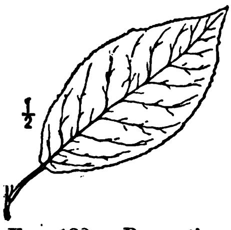


FIG. 188.—*P. serotina.*

239. Chestnut.

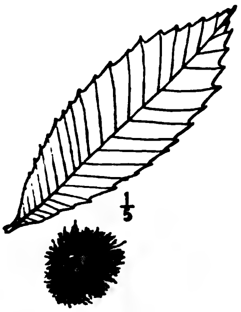


FIG. 189.—*C. vulgaris.*

58. CHESTNUT (*Castanea vulgaris* var. *americana*): Wood light, moderately soft, stiff, not strong, of coarse texture; the sapwood light, the heartwood darker brown. It shrinks and checks considerably in drying, works easily, stands well, and is very durable. Used in cabinetwork, cooperage, for railway-ties, telegraph-poles, and locally in heavy construction. Medium-sized tree, very common in the Alleghanies, occurs from Maine to Michigan and southward to Alabama.

59. CHINQUAPIN (*Castanea pumila*): A small-sized tree, with wood slightly heavier than, but otherwise similar to, the preceding; most common in Arkansas, but with nearly the same range as the chestnut.

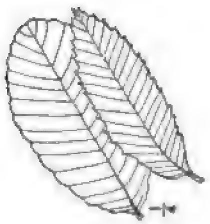


FIG. 190.—*C. pumila.*

Fig. 191.—*C. chrysophylla.*

60. Chinquapin (*Castanopsis chrysophylla*): A medium-sized tree of the western ranges of California and Oregon.

240. Coffee-tree.

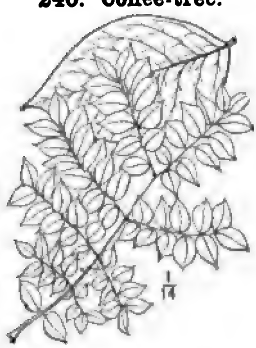


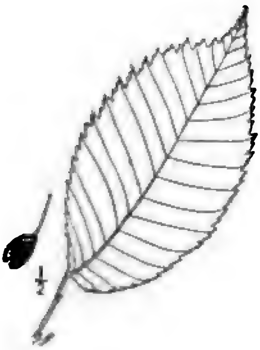
Fig. 192.—*G. canadensis.*

61. Coffee-tree (*Gymnocladus canadensis*) (coffee-nut): Wood heavy, hard, strong, very stiff, of coarse texture; durable; the sapwood yellow, the heartwood reddish brown; shrinks and checks considerably in drying; works and stands well and takes a good polish. It is used to a limited extent in cabinetwork. A medium to large-sized tree; not common. Pennsylvania to Minnesota and Arkansas.

Cottonwood. See Poplar.

Cucumber-tree. See Tulip.

241. Elm.—Wood heavy, hard, strong, very tough; moderately durable in contact with the soil; commonly cross-grained, difficult to split and shape, warps, and checks considerably in drying, but stands well if properly handled. The broad sapwood whitish, heart brown, both with shades of gray and red; on split surface rough; texture coarse to fine; capable of high polish. Elm is used in the construction of cars, wagons, etc., in boat- and ship-building, for agricultural implements and machinery; in rough cooperage, saddlery and harness work, but particularly in the manufacture of all kinds of furniture, where the beautiful figures, especially those of the tangential or bastard section, are just beginning to be duly appreciated. The elms are medium to large-sized trees, of fairly rapid growth, with stout trunk, form no forests of pure growth, but are found scattered in all the broad-leaved woods of our country, sometimes forming a considerable portion of the arborescent growth.



62. WHITE-ELM (*Ulmus americana*) (American elm, water-elm): Medium to large-sized tree, common. Maine to Minnesota, southward to Florida and Texas.

FIG. 193.—*U. americana.*

63. ROCK-ELM (*Ulmus racemosa*) (cork-elm, hickory-elm, white elm, cliff-elm): Medium to large-sized tree. Michigan, Ohio, from Vermont to Iowa, southward to Kentucky.

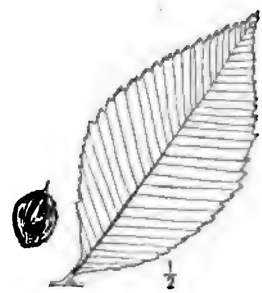
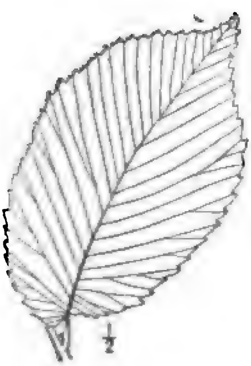


FIG. 194.—*U. racemosa.*



64. RED ELM (*Ulmus fulva*) (slippery elm, moose-elm): Small-sized tree, found chiefly along watercourses. New York to Minnesota, and southward to Florida and Texas.

FIG. 195.—*U. fulva.*

65. CEDAR-ELM (*Ulmus crassifolia*): Small-sized tree, quite common. Arkansas and Texas.

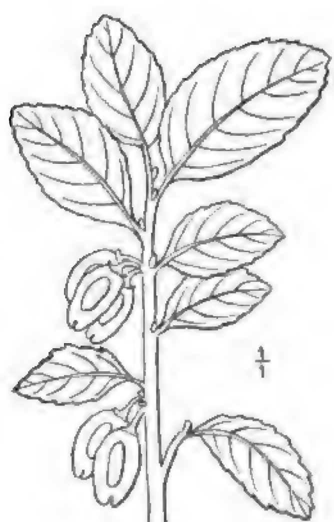


FIG. 196.—*U. crassifolia*

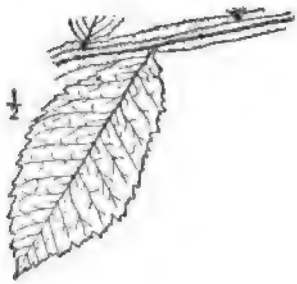


Fig. 197.—*U. alata.*

66. Winged Elm (*Ulmus alata*) (Wahoo): Small-sized tree, locally quite common. Arkansas, Missouri, and eastern Virginia.

242. Gum.—This general term refers to two kinds of wood usually distinguished as sweet or red gum, and sour, black, or tupelo gum, the former being a relative of the witch-hazel, the latter belonging to the dogwood family.

Fig. 198.—*N. sylvatica.*

67. Tupelo (*Nyssa sylvatica*) (sour gum, black gum): Maine to Michigan, and southward to Florida and Texas. Wood heavy, hard, strong, tough, of fine texture, frequently cross-grained, of yellowish or grayish-white color, hard to split and work, troublesome in seasoning, warps and checks considerably, and is not durable if exposed; used for wagon-hubs, wooden ware, handles, wooden shoes, etc. Medium to large-sized trees, with straight, clear trunks; locally quite abundant, but never forming forests of pure growth.

68. Tupelo Gum (*Nyssa uniflora*) (cotton-gum): Lower Mississippi basin, northward to Illinois and eastward to Virginia; otherwise like preceding species.

Fig. 199.—*N. uniflora.*

69. SWEET GUM (*Liquidambar styraciflua*) (red gum, liquidambar, bilsted): Wood rather heavy, rather soft, quite stiff and strong, tough, commonly cross-grained, of fine texture; the broad sapwood whitish, the heart-wood reddish brown; the wood shrinks and warps considerably, but does not check badly, stands well when fully seasoned, and takes good polish. Sweet gum is used in carpentry, in the manufacture of furniture, for cut veneer, for wooden plates, plaques, baskets, etc., also for wagon-hubs, hat-blocks, etc. A large-sized tree, very abundant, often the principal tree in the swampy parts of the bottoms of the Lower Mississippi Valley; occurs from New York to Texas, and from Indiana to Florida.

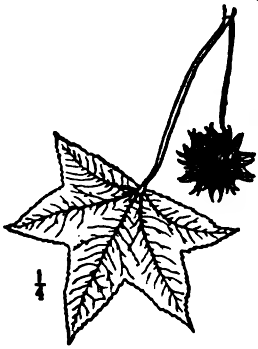


FIG. 200.—*L. styraciflua.*

243. Hackberry.

70. HACKBERRY (*Celtis occidentalis*) (sugar-berry): The handsome wood heavy, hard, strong, quite tough, of moderately fine texture, and greenish- or yellowish-white color; shrinks moderately, works well, and takes a good polish. So far but little used in the manufacture of furniture. Medium to large-sized tree, locally quite common, largest in the Lower Mississippi Valley; occurs in nearly all parts of the Eastern United States.

FIG. 201.—*C. occidentalis.*

244. Hickory.—Wood very heavy, hard, and strong, proverbially tough, of rather coarse texture, smooth and of straight grain. The broad sapwood white, the heart reddish nut-brown. It dries slowly, shrinks and checks considerably; is not durable in the ground, or if exposed, and especially the sapwood, is always subject to the inroads of boring-insects. Hickory excels as carriage and wagon stock, but is also extensively used in the manufacture of implements and machinery, for tool-handles, timber-pins, for harness work and cooperage. The hickories are tall trees with slender stems, never form forests, occasionally small groves, but usually occur scattered among other broad-leaved trees in suitable localities. The following species all contribute more or less to the hickory of the markets:

FIG 202.—*H. ovata.*

71. SHAGBARK HICKORY (*Hicoria ovata*) (shellbark hickory): A medium to large-sized tree, quite common; the favorite among hickories; best developed in the Ohio and Mississippi basins; from Lake Ontario to Texas, Minnesota to Florida.

77. Mockernut Hickory (*Hicoria alba*) (black hickory, bull- and black-nut, big-bud, and white-heart hickory): A medium to large-sized tree, with the same range as the foregoing; common, especially in the South.

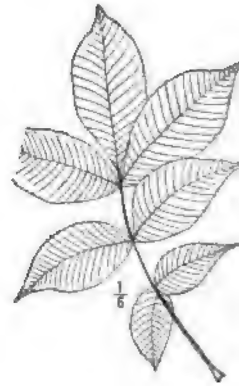


Fig. 203.—*H alba.*

Fig. 204.—*H. glabra.*

73. Pignut Hickory (*Hicoria glabra*) (brown hickory, black hickory, switch-bud hickory): Medium to large-sized tree, abundant; all Eastern United States.

74. Bitter-nut Hickory (*Hicoria minima*) (swamp hickory): A medium-sized tree, favoring wet localities, with the same range as the preceding.

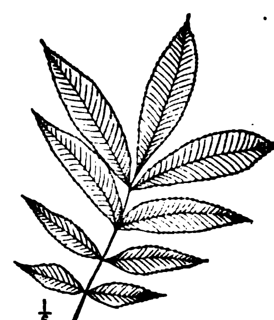


Fig. 205.—*H. minima.*

Fig. 206.—*H. pecan.*

75. Pecan (*Hicoria pecan*) (Illinois nut): A large tree, very common in the fertile bottoms of the Western streams. Indiana to Nebraska and southward to Louisana and Texas.

245. Holly.

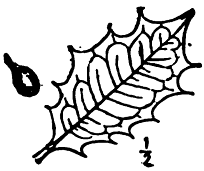


FIG. 207.—*I. opaca.*

76. HOLLY (*Ilex opaca*): Wood of medium weight, hard, strong, tough, of fine texture and white color; works and stands well, used for cabinetwork and turnery. A small tree, most abundant in the Lower Mississippi Valley and Gulf States, but occurring eastward to Massachusetts and north to Indiana.

Horse-chestnut. See BUCKEYE.

Ironwood. See BLUE BEECH.

246. Locust.—This name applies to both of the following:

77. BLACK LOCUST (*Robinia pseudacacia*) (black locust, yellow locust): Wood very heavy, hard, strong, and tough, of coarse texture, very durable in contact with the soil, shrinks considerably, and suffers in seasoning; the very narrow sapwood yellowish, the heartwood brown, with shades of red and green. Used for wagon-hubs, treenails or pins, but especially for ties, posts, etc. Abroad it is much used for furniture and farm-implements, and also in turnery. Small to medium-sized tree, at home in the Alleghanies, extensively planted, especially in the West.

FIG. 208.—*R. pseudacacia.*

78. HONEY-LOCUST (*Gleditschia triacanthos*) (black locust, sweet locust, three-thorned acacia): Wood heavy, hard, strong, tough, of coarse texture, susceptible of a good polish, the narrow sapwood yellow, the heartwood brownish red. So far but little appreciated except for fencing and fuel; used to some extent for wagon-hubs and in rough construction. A medium-sized tree, found from Pennsylvania to Nebraska, and southward to Florida and Texas; locally quite abundant.

FIG. 209.—*G. triacanthos.*

Magnolia. See TULIP.

247. Maple.—Wood heavy, hard, strong, stiff, and tough, of fine texture, frequently wavy-grained, this giving rise to "curly" and "blister" figures; not durable in the ground or otherwise exposed. Maple is creamy white, with shades of light brown in the heart; shrinks moderately, seasons, works and stands well, wears smoothly, and takes a fine polish. The wood is used for ceiling, flooring, panelling, stairway, and other finishing-lumber in house, ship, and car construction; it is used for the keels of boats and ships, in the manufacture of implements and machinery, but especially for furniture, where entire chamber sets of maple rival those of oak. Maple is also used

for shoe-lasts and other form-blocks, for shoe-pegs, for piano actions, school apparatus, for wood type in show-bill printing, tool-handles, in wood-carving, turnery, and scrollwork. The maples are medium-sized trees, of fairly rapid growth; sometimes form forests and frequently constitute a large proportion of the arborescent growth.

79. SUGAR-MAPLE (*Acer saccharum*) (hard maple, rock-maple): Medium to large-sized tree, very common, forms considerable forests. Maine to Minnesota, abundant, with birch, in parts of the pineries; southward to northern Florida; most abundant in the region of the Great Lakes.

FIG. 210.—*A. saccharum.*

80. RED MAPLE (*Acer rubrum*) (swamp- or water-maple): Medium-sized tree. Like the preceding, but scattered along watercourses and other moist localities.

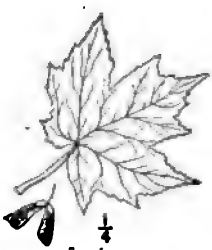


FIG. 211.—*A. rubrum.*

81. SILVER MAPLE (*Acer saccharinum*) (soft maple, silver maple): Medium-sized, common; wood lighter, softer, inferior to hard maple, and usually offered in small quantities and held separate in the market. Valley of the Ohio, but occurs from Maine o Dakota, and southward to Florida.

FIG. 212.—*A. saccharinum.*

82. BROAD-LEAVED MAPLE (*Acer macrophyllum*): Medium-sized tree, forms considerable forests, and like the preceding has a lighter, softer, and less valuable wood. Pacific Coast.

FIG. 213.—*A. macrophyllum.*

248. Mulberry.

83. Red Mulberry (*Morus rubra*): Wood moderately heavy, hard, strong, rather tough, of coarse texture, durable; sapwood whitish, hard yellow to orange-brown; shrinks and checks considerably in drying; works and stands well. Used in cooperage and locally in ship-building and in the manufacture of farm-implements. A small-sized tree, common in the Ohio and Mississippi valleys, but widely distributed in the Eastern United States.


$\frac{1}{4}$

Fig. 214.—*M. rubra.*

249. Oak.—Wood very variable, usually very heavy and hard, very strong and tough, porous, and of coarse texture; the sapwood whitish, the heart "oak" brown to reddish brown. It shrinks and checks badly, giving trouble in seasoning, but stands well, is durable, and little subject to attacks of insects. Oak is used for many purposes: in ship-building, for heavy construction, in common carpentry, in furniture, car, and wagon work, cooperage, turnery, and even in wood-carving; also in the manufacture of all kinds of farm-implements, wooden mill machinery, for piles and wharves, railway-ties, etc. The oaks are medium to large-sized trees, forming the predominant part of a large portion of our broad-leaved forests, so that these are generally "oak forests," though they always contain a considerable proportion of other kinds of trees. Three well-marked kinds, white, red, and live oak, are distinguished and kept separate in the market. Of the two principal kinds white oak is the stronger, tougher, less porous, and more durable. Red oak is usually of coarser texture, more porous, often brittle, less durable, and even more troublesome in seasoning than white oak. In carpentry and furniture work red oak brings about the same price at present as white oak. The red oaks everywhere accompany the white oaks, and, like the latter, are usually represented by several species in any given locality. Live-oak, once largely employed in ship-building, possesses all the good qualities (except that of size) of white oak even to a greater degree. It is one of the heaviest, hardest, and most desirable building-timbers of this country; in structure it resembles the red oaks, but is much less porous.


$\frac{1}{4}$

Fig. 215.—*Q. alba.*

84. White Oak (*Quercus alba*): Medium to large-sized tree, common in the Eastern States, Ohio and Mississippi valleys; occurs throughout Eastern United States.

85. BUR-OAK (*Quercus macrocarpa*) (mossy-cup oak, over-cup oak): Large-sized tree, locally abundant, common. Bottoms west of Mississippi; range farther west than preceding.

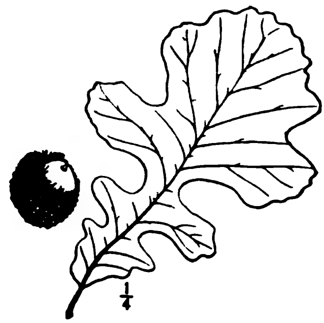
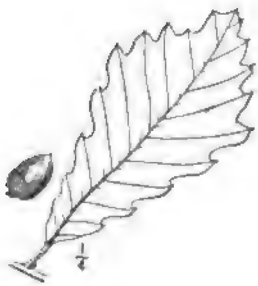


FIG. 216.—*Q. macrocarpa.*



86. SWAMP WHITE OAK (*Quercus bicolor*): Large-sized tree, common. Most abundant in the Lake States, but with range as in white oak.

FIG. 217.—*Q. bicolor.*

87. YELLOW OAK (*Quercus prinoides*) (chestnut-oak, chinquapin oak): Medium-sized tree. Southern Alleghanies, eastward to Massachusetts.

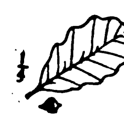
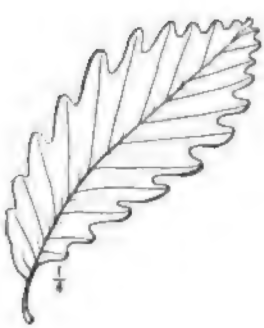


FIG. 218.
Q. prinoides.



88. BASKET-OAK (*Quercus michauxii*) (cow-oak): Large-sized tree, locally abundant; lower Mississippi and eastward to Delaware.

FIG. 219.—*Q. michauxii.*

89. OVER-CUT OAK (*Quercus lyrata*) (swamp white oak, swamp post-oak): Medium to large-sized tree, rather restricted; ranges as in the preceding.

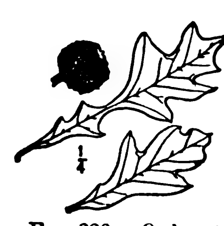


FIG. 220.—*Q. lyrata.*

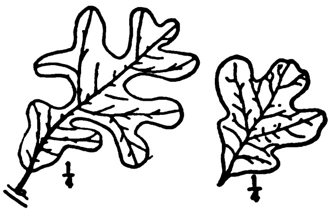


Fig. 221.—*Q. obtusiloba.*

90. Post-oak (*Quercus obtusiloba*) (iron-oak): Medium to large-sized tree. Arkansas to Texas, eastward to New England, and northward to Michigan.

Fig. 222. — *Q. durandii.*

91. White Oak (*Quercus durandii*): Medium to small-sized tree. Texas, eastward to Alabama.

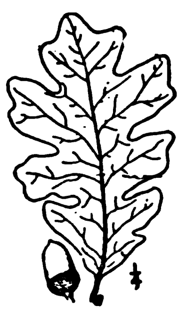


Fig. 223.—*Q. garryana.*

92. White Oak (*Quercus garryana*): Medium to large-sized tree. Washington to California.

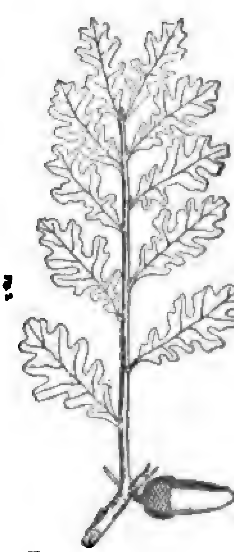


Fig. 224.—*Q. lobata.*

93. White Oak (*Quercus lobata*): Medium to large-sized tree; largest oak on the Pacific coast. California.

FIG. 225.—*Q. rubra.*

94. RED OAK (*Quercus rubra*) (black oak): Medium to large-sized tree; common in all parts of its range. Maine to Minnesota, and southward to the Gulf.

FIG. 226.—*Q. tinctoria.*

95. BLACK OAK (*Quercus tinctoria*) (yellow oak): Medium to large-sized tree; very common in the Southern States, but occurring north as far as Minnesota, and eastward to Maine.

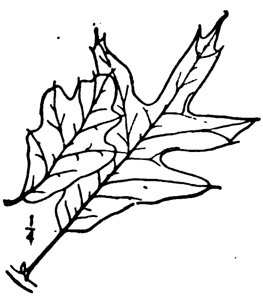


FIG. 227.—*Q. falcata.*

96. SPANISH OAK (*Quercus falcata*) (red oak): Medium-sized tree; common in the South Atlantic and Gulf region, but found from Texas to New York, and north to Missouri and Kentucky.

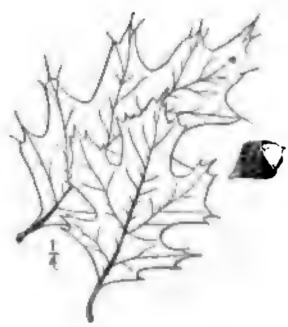


FIG. 228.—*Q. coccinea.*

97. SCARLET OAK (*Quercus coccinea*): Medium to large-sized tree; best developed in the lower basin of the Ohio, but found from Maine to Missouri, and from Minnesota to Florida.

FIG. 229.—*Q. palustris.*

98. PIN-OAK (*Quercus palustris*) (swamp Spanish oak, water-oak): Medium to large-sized tree, common along borders of streams and swamps. Arkansas to Wisconsin, and eastward to the Alleghanies.

99. WILLOW-OAK (*Quercus phellos*) (peach-oak): Small to medium-sized tree. New York to Texas, and northward to Kentucky.

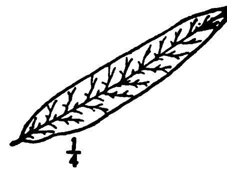


FIG. 230.—*Q. phellos.*

FIG. 231.—*Q. aquatica.*

100. WATER-OAK (*Quercus aquatica*) (duck-oak, possum-oak, punk-oak): Medium to large-sized tree, of extremely rapid growth. Eastern Gulf States, eastward to Delaware, and northward to Missouri and Kentucky.

101. LIVE-OAK (*Quercus virens*): Small-sized tree, scattered along the coast from Virginia to Texas.

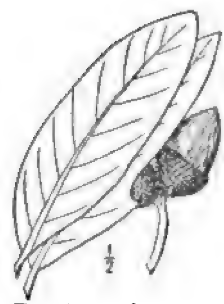


FIG. 232.—*Q. virens.*

102. LIVE-OAK (*Quercus chrysolepis*) (maul-oak, Valparaiso oak): Medium-sized tree. California.

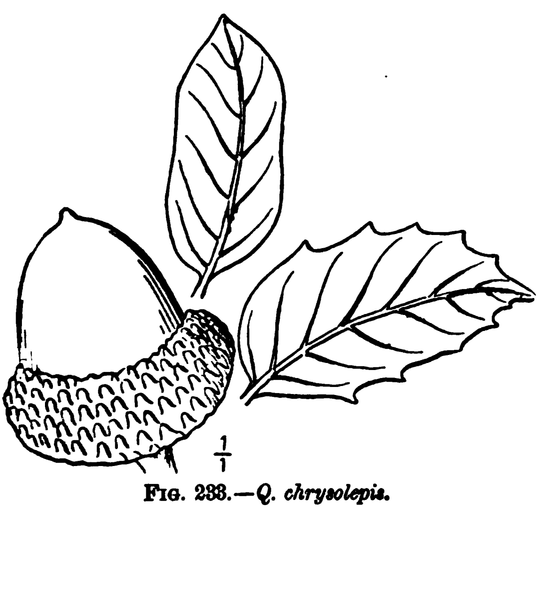


FIG. 233.—*Q. chrysolepis.*

250. Osage Orange.

103. OSAGE ORANGE (*Maclura aurantiaca*) (Bois d'Arc): Wood very heavy, exceedingly hard, strong, not tough, of moderately coarse texture, and very durable; sapwood yellow, heart brown on the end, yellow on longitudinal faces, soon turning grayish brown if exposed; it shrinks considerably in drying, but once dry it stands unusually well. Formerly much used for wheel stock in the dry regions of Texas; otherwise employed for posts, railway-ties, etc. Seems too little appreciated; it is well suited for turned ware and especially for wood-carving. A small-sized tree, of fairly rapid growth, scattered through the rich bottoms of Arkansas and Texas.

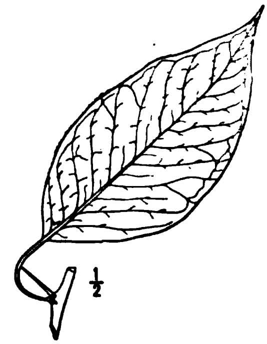


FIG. 234.—*M. aurantiaca.*

251. Persimmon.

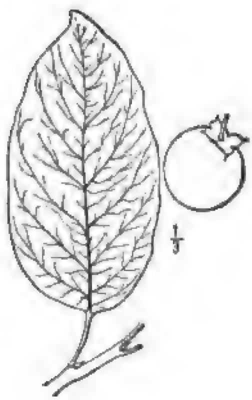


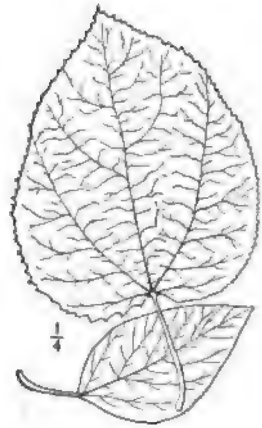
FIG. 235.—*D. virginiana.*

104. PERSIMMON (*Diospyros virginiana*): Wood very heavy and hard, strong and tough; resembles hickory, but is of finer texture; the broad sapwood cream-color, the heart black; used in turnery for shuttles, plane-stocks, shoe-lasts, etc. Small to medium-sized tree, common and best developed in the Lower Ohio Valley, but occurs from New York to Texas and Missouri.

252. Poplar and Cottonwood. (See also TULIP-WOOD.)—Wood light, very soft, not strong, of fine texture and whitish, grayish, to yellowish color, usually with a satiny lustre. The wood shrinks moderately (some cross-grained forms warp excessively), but checks little; is easily worked, but is not durable. Used as building- and furniture-lumber, in cooperage for sugar- and flour-barrels, for crates and boxes (especially cracker-boxes), for woodenware and paper-pulp.

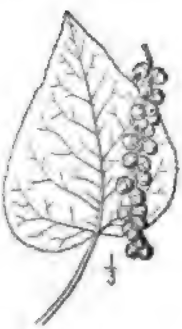
105. COTTONWOOD (*Populus monilifera*): Large-sized tree; forms considerable forests along many of the Western streams, and furnishes most of the cottonwood of the market. Mississippi Valley and west; New England to the Rocky Mountains.

FIG. 236.—*P. monilifera.*



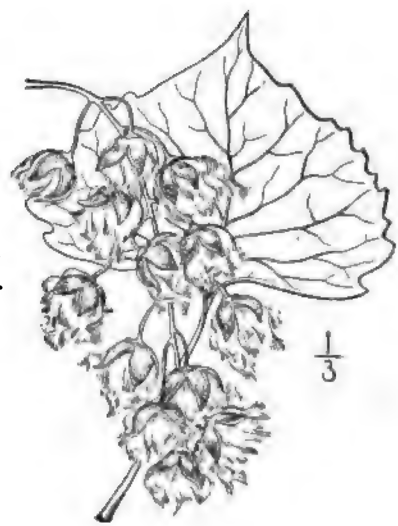
106. BALSAM (*Populus balsamifera*) (balm of Gilead): Medium to large-sized tree; common all along the northern boundary of the United States.

FIG. 237.—*P. balsamifera.*



107. BLACK COTTONWOOD (*Populus trichocarpa*): The largest deciduous tree of Washington; very common. Northern Rocky Mountains and Pacific region.

FIG. 238.
P. trichocarpa.



108. COTTONWOOD (*Populus fremontii* var. *wislizeni*): Medium to large-sized tree, common. Texas to California.

FIG. 239.—*P. wislizeni.*

109. POPLAR (*Populus grandidentata*): Medium-sized tree, chiefly used for pulp. Maine to Minnesota and southward along the Alleghanies.

FIG. 240.
P. grandidentata.

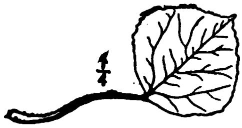


FIG. 241.
P. tremuloides.

110. ASPEN (*Populus tremuloides*): Small to medium-sized tree, often forming extensive forests and covering burned areas. Maine to Washington and northward, south in the Western mountains to California and New Mexico.

Sour Gum. See GUM.

Red Gum. See GUM.

253. Sassafras.

111. SASSAFRAS (*Sassafras sassafras*): Wood light, soft, not strong, brittle, of coarse texture, durable; sapwood yellow, heart orange-brown. Used in cooperage, for skiffs, fencing, etc. Medium-sized tree, largest in the Lower Mississippi Valley, from New England to Texas, and from Michigan to Florida.

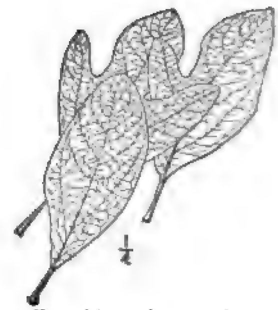


FIG. 242.—*S. sassafras.*

Sweet Gum. See GUM.

254. Sycamore.

112. SYCAMORE (*Platanus occidentalis*) (button-wood, buttonball-tree, water-beech): Wood moderately heavy, quite hard, stiff, strong, tough, usually cross-grained, of coarse texture, and white to light-brown color; the wood is hard to split and work, shrinks moderately, warps and checks considerably, but stands well. It is used extensively for drawers, backs, bottoms, etc., in cabinetwork, for tobacco-boxes, in cooperage, and also for finishing lumber, where it has too long been underrated. A large tree, of rapid growth, common and largest in the Ohio and Mississippi valleys, at home in nearly all parts of the Eastern United States. The California species—

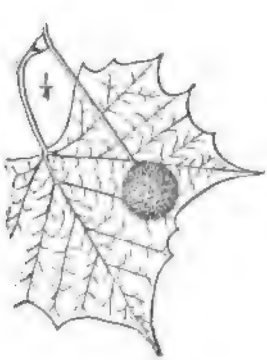


FIG. 243.—*P. occidentalis.*

113. *Platanus racemosa* — resembles in its wood the Eastern form.

255. Tulip-wood.

114. TULIP-TREE (*Liriodendron tulipifera*) (yellow poplar, white wood): Wood quite variable in weight, usually light, soft, stiff but not strong, of fine texture and yellowish color; the wood shrinks considerably, but seasons without much injury; works and stands remarkably well. Used for siding, for panelling and finishing-lumber in house-, car-, and ship-building, for sideboards and panels of wagons and carriages; also in the manufacture of furniture, implements, and machinery, for pump-logs, and almost every kind


FIG. 244.—*L. tulipifera.*

of common woodenware, boxes, shelving, drawers, etc. An ideal wood for the carver and toyman. A large tree, does not form forests, but is quite common, especially in the Ohio Basin; occurs from New England to Missouri and southward to Florida.

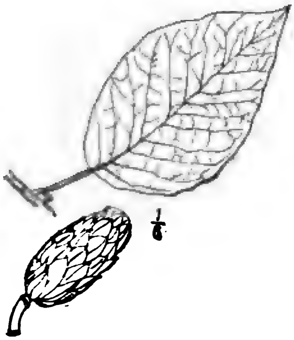


FIG. 245.—*M. acuminata.*

115. CUCUMBER-TREE (*Magnolia acuminata*): A medium-sized tree, most common in the southern Alleghanies, but distributed from New York to Arkansas, southward to Alabama, and northward to Illinois. Resembling, and probably confounded with, tulip-wood in the markets.

Tupelo. See GUM.

256. Walnut.

116. BLACK WALNUT (*Juglans nigra*): Wood heavy, hard, strong, of coarse texture; the narrow sapwood whitish, the heartwood chocolate-brown. The wood shrinks moderately in drying, works and stands well, takes a good polish, is quite handsome, and has been for a long time the favorite cabinet-wood in this country. Walnut, formerly used even for fencing, has become too costly for ordinary uses, and is to-day employed largely as a veneer, for inside finish and cabinetwork; also in turnery, for gunstocks, etc. Black walnut is a large tree, with stout trunk, of rapid growth, and was formerly quite abundant throughout the Alleghany region, occurring from New England to Texas, and from Michigan to Florida.

FIG. 246.—*J. nigra.*

White Walnut. See BUTTERNUT

White Wood. See TULIP, and also BASSWOOD.

Yellow Poplar. See TULIP.

PART III.

TESTING-MACHINES AND METHODS OF TESTING MATERIALS OF CONSTRUCTION.

CHAPTER XIV.

MECHANICAL TESTS IN GENERAL.

257. General Observations.—Mechanical tests are those most commonly used to discover the working qualities of the materials of construction. Since these materials nearly always have to resist the action of external forces, it follows that the suitableness of such a material to resist the action of these forces is best determined by tests approximating as nearly as may be to the conditions of actual practice.

Mechanical tests, therefore, are of supreme importance in the study of any building material. By standardizing the conditions under which these tests are carried out, the results become comparable wherever or by whomsoever they are made, and they also become authoritative in all countries and for all purposes. If such results can be made wholly independent of the means employed in making the tests, and hence to furnish a knowledge of the true characteristics of the material, they can be used safely in theoretical generalizations on the one hand, and in the practical designing of structures on the other. With many kinds of tests this ideal divorcement of the results from the conditions of the tests can certainly never be attained, as in the case of tests by impact, but it doubtless can be practically attained in some of the more simple tests, as in tension and compression. In the former case the most that can be accomplished is to prescribe uniform conditions in order that the results obtained by different experimenters may be comparable, although they may not serve for accurate scientific generalizations. They might also serve to give a relative value to the various materials or samples so tested, and to grade them with some degree of approximation to their true relative merits for a proposed purpose. Such tests, therefore, may serve fully their immediate object even though the

results can be given no absolute significance whatever. If, however, the conditions of such tests are allowed to vary, they would lose even this relative significance, and would therefore be quite worthless. The standardizing of any particular kind of test evidently depends on the state of the science at the time; and as our knowledge of any particular property of a material increases, it is probable that our standard methods of testing will also have to change. No such standards, therefore, can be fixed permanently, but certain methods can be agreed on and followed for a time, and when a change is made let all change together. To attain to this kind of unity of action it is necessary to have a world's representative body which will command the confidence and allegiance of both the theoretical and the practical users of materials in all civilized countries to decide such questions. A beginning has been made in this direction in the International Commission on the Standardization of Methods of Testing the Materials of Construction, which has had several meetings in Europe at intervals of about three years, the last one being at Zurich in September, 1895, where a permanent organization was effected. The French Government, also (in 1891, as a result of action taken by engineers at their centennial exposition in 1889), appointed a national French Commission of over one hundred of the leading authorities in France to report on this subject. Their report, printed in four quarto volumes (1895), is to-day (1897) by far the best single source of information on these subjects. They have proposed what appeared to them practicable standard tests for nearly all kinds of structural materials.

Evidently no complete standardization can be effected for tests on entire structural forms, since these vary in shape, size, and disposition of parts, but specimen tests can be standardized since all significant conditions can be made uniform.

258. Mechanical Tests Classified.—In a general way we may divide mechanical tests of building materials into the following classes:

With reference to the method of applying the loads we have—

(1) *Static Tests*, or those made with gradually increasing loads, such as the ordinary tests in tension, compression, cross-bending, torsion, and shearing.

(2) *Dynamic Tests*, or those made with suddenly applied loads, as by a falling weight.

(3) *Wearing Tests*, or those made for determining resistance to abrasion and impact, as in the case of paving-materials.

With reference to the character of the test specimen we have—

(1) *Specimen Tests*, or those made upon specimens of the material specially prepared and given standard forms and dimensions.

(2) *Structural Tests*, or those made on full-sized structural forms, as bridge members, brick piers, pipes, wire ropes, chains, riveted joints, etc., or on the structure as a whole, such as boilers, simple trusses, frames, and various parts of machines.

Complete standard rules for making tests of structural materials can be

adopted for making all kinds of tests on specially prepared specimens, but they can be only partially prescribed for tests of structural forms.

259. General Remarks on Testing-machines.—The following considerations apply to testing-machines and testing-appliances in general:

1. The weighing apparatus should be quite independent of the loading apparatus, the former usually being fixed and the latter movable.

2. In lever machines the length of the knife-edges must be proportioned to the maximum loads in order not to be crushed down, and they should be so placed that all will receive their share of the load. They must also be so mounted as not to change the leverage by any reaction displacement which may occur To insure this, the knife-edges must be attached to the levers, and the bearings to the platform.

3. The knife-edges and bearings of any beam must lie in the same straight line, and this line should lie in the gravity axis of the beam and its rigid attachments. This is especially necessary for the weighing-beam itself, so that its vertical angular movement may not disturb the counterbalancing. If the poise is moved by a cord over a pair of pulleys, this cord should be attached to the poise-hanger in this same axial line, so that the pulling of the poise may not supply a leverage on the beam to raise or lower it.

4. Manometer machines have many peculiar errors. For example, any air-bubble in the indicating liquid vitiates the results by its own change in volume under pressure. Again, the exact area of surface subjected to pressure is always uncertain.

5. The weighing apparatus should be so constructed as to be readily verified by the imposition of known weights, and the parts should be open to inspection and easily repaired and kept in order.

6. A precision of 1 in 250 has been considered sufficient.* This is a proportional error of 0.4 of 1 per cent.

7. The loading should proceed gradually and uniformly, and not by sudden increments as by large pump-pulsations, or by the adding of over-weights by hand to the weighing-beam. The rate of loading should also be under perfect control.

8. The machine should be so constructed as to permit the free use of appliances for measuring distortion of the specimen by some suitable device.

9. If used for compression tests, one of the bearing-surfaces should be slightly adjustable to accommodate the machine to the non-parallel faces of the test-block, and these bearing-surfaces should be harder than any material tested by them. The neutral axis of these bearing-plates should coincide with the axis of symmetry of the applied forces as transmitted by the machine to the specimen. For these tests the machine should have a very slow movement.

10. If used for cross-breaking tests, it should be furnished with means for measuring the deflection. To do this properly a rigid connection must be

* This standard is given by the French Commission.

established between the two end bearings to the middle bearing (or bearings), *through parts not under stress*, in order that the loading of the specimen may not disturb this rigid relation.

11. Torsion testing-machines should apply the torsion movement as a true couple and without developing any tensile or bending stress in the specimen.

12. Impact testing-machines should as far as possible satisfy the conditions imposed in Art. 292. That is, as far as possible, the entire energy of the blow should pass into the specimen. The falling weight should be held to its course either by vertical guides, in the case of a falling weight, or by a pendulum mounted on a transverse axis resting on knife-edge bearings. The former method is to be preferred. The falling weight should be symmetrical in form, with suitable guiding attachments, to be formed (cast) in one piece, of hard metal, with its centre of gravity as low as possible. The height of the weight should be greater than the width between the guides, which latter should be quite rigid, true, and vertical, and should offer no frictional resistance to the falling weight. The supporting mass should be very great as compared to that of the falling weight. The French Commission recommend that it be at least 15 or 20 times that of the striking body. Impact tests can only be standardized by using exactly similar appliances in all respects, including the supporting blocks and the foundation on which these supports rest.

260. The Effect of the Rate of Loading on the Results of the Test.—The French Commission quote M. A. Le Chatelier on this subject as follows: "Metals do not respond instantly to the deforming action of external forces. These deformations, both elastic and permanent, continue to increase with time, and the termination of the instant when the deformation corresponding to a given load has been fully completed depends only on the exactness of the measuring instruments employed. Speaking absolutely, this condition of equilibrium is never attained, and we may say the deformation increases indefinitely. It approaches, however, a limiting value (as an asymptote), especially in the case of elastic deformations, and even for permanent deformations the time may be found beyond which the remaining deformation will not exceed a given amount."

It is admitted, however, that for metals at ordinary temperatures a tension test (for instance), extended over a few minutes' time, gives practically the same results, in every respect, that would be obtained by any slower imposition of the load. This has been thoroughly established by Bauschinger, as well as by Considère and Le Chatelier. Zinc and tin are exceptions to this law, comparatively small external forces causing final rupture if these forces continue active. Copper and aluminum also fail under a somewhat smaller permanent load than is required to produce rupture in ordinary tests.* It is well known that timber yields continually under about one half

* Report of the French Commission, vol. I. p. 93.

the breaking-load, and this half-load, permanently placed, may ultimately cause failure. For all kinds of test specimens of wrought iron and steel, and other structural metals, at ordinary temperatures, a test extending to one minute or more may be considered as giving normal results.

For very rapid tests, or where rupture occurs in less than a minute, the breaking-load increases and so does the ultimate elongation. In the case of soft steel, however, which has a great local reduction of area, the elongation diminishes as the test period decreases, reaching a minimum for a period of about one minute, and then the elongation rapidly increases again for very quick tests, because it then reduces in cross-section more uniformly throughout its entire length.

It has been shown by M. Considère * that the stress-diagram giving the simultaneous relation between stress and deformation is very different under very quick impositions of load, as in case of a shock, or impact, from the diagram for ordinary laboratory static tests of more than one minute duration. His results are shown in Fig. 52, p. 79. This subject is fully discussed in Art. 292, where impact tests are described.

In conclusion it may be said that in all ordinary tests of metals the test period should fall between one and six minutes.

261. Significant Limits of Deformation.—Ever since the properties of building materials have been studied, " the elastic limit " has been defined both as the greatest load which will not produce a permanent set, and as the greatest load at which deformation remains proportional to the load, or stress. It has commonly been supposed that these two limits were one and the same, and in the commercial testing, which has probably been carried on in America to as great an extent as in any country, this so-called " elastic limit " has been observed as the point at which the deformation increases rapdly under a constant load, or, as it has been called, the " yield-point." The French Commission has studied this subject with care, and after mature deliberation a majority have agreed to adopt three critical points, as follows:

1. *The elastic limit* (" *la limite d'élasticité* "), or the unit stress beyond which a portion of the deformation remains as a *permanent set.* (Point E of the stress-diagram.)

2. *The proportional elastic limit* (*la limite d'élasticité proportionelle*, or *limite des déformations proportionelles*), corresponding to the point where the deformation ceases to be proportional to the loads. (Point P of the stress-diagram.)

3. *The apparent elastic limit* (*la limite d'élasticité apparente*, or *origine des déformations sous charge constante*), corresponding to the point where the deformations increase rapidly without any increase in the force exerted. (Point F of the stress-diagram.)

While in a scientific study of metals it may be important to determine all these three limits whenever they occur, in practical or commercial testing

* French Commission Report, vol. II. p. 344.

it will be found sufficient to observe the third one only, or the second only in case the third limit does not obtain for that material. The first of these limits has seldom been determined, since it involves a release of the stress by removing the load. This involves a great loss of time, since to determine this limit accurately the load would have to be released for each small increment of, let us say, 1000 pounds per square inch for iron and steel. The second and third limits can be determined after the test has been completed, provided simultaneous readings of the load and deformation were taken during the test at frequent intervals, or provided an autographic stress-diagram was made by suitable attachments to the machine and test specimen. The third limit is commonly, but not accurately, determined in commercial testing in America, by "the drop of the weighing-beam," this marking the point where the deformation increases for very ductile metals under a constant load.

With unrolled (or unforged) castings of the various metals any load produces some appreciable permanent set, so that the first elastic limit does not exist for such materials. It is probable that this is also true to an inappre-

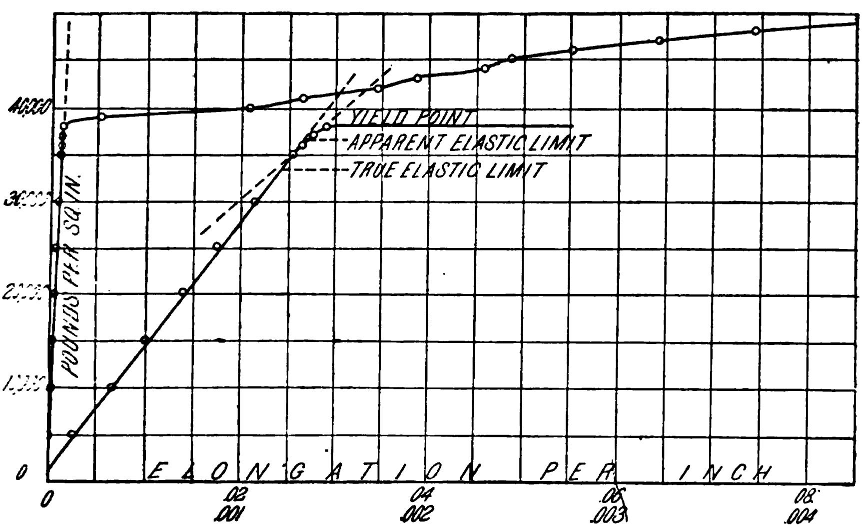


FIG. 247.—Average Curve of Four Tests of ¾-in. Wrought-iron Rods. (*Wat. Ars. Rep.*, 1888.)

ciable degree of all the rolled metals, so that this is a very unreliable and unsatisfactory test of any important property of materials. The law of proportionality is much better defined, but, owing to the mixed character of the elementary forms entering into the composition of all metal products, even the purest, this law is often found to fail when the most refined means of measurement are employed, since there then seems to be no strictly constant ratio between the load-increments and the corresponding increments of the deformation, and hence the exact point where this ratio begins to

change is in these cases difficult to fix. In such cases, by plotting the deformation to a very large scale with the loads, one can determine graphically, as in Fig. 247, about where the deformation increments begin to increase. The second and third elastic limits are marked on this diagram "true elastic limit" and "yield-point" respectively. The "apparent elastic limit," or "yield-point," is the most important and significant of the three, as well as the most easily determined, but in high carbon-steel, especially in hard-steel wire and in all cast metals, and often in wrought iron, this point does not appear, in which case the "proportional elastic limit" takes its place, as shown in Fig. 249, where it is marked as the

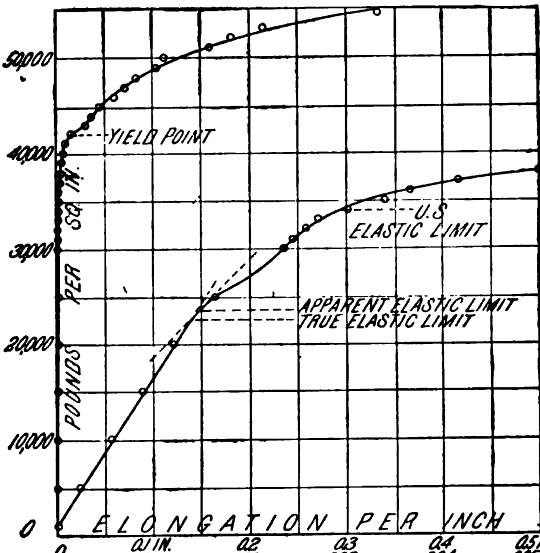


Fig. 248.—Typical Stress-diagram of Hard-drawn Brass. (*Wat. Ars. Rep.*, **1886.**) **(The "U. S. Elastic Limit" is that given in the published report.)**

"true elastic limit." Sometimes, also, when point F is very marked, point P is found above it, as in Figs. 7 and 8, pp. 15 and 16.

262. All these Absolute Elastic Limits Unsatisfactory.—It is proposed now to show that no one of the three definitions of elastic limit given in Art. 261 can be used in practice. They all will be shown to be either absolutely indeterminate or wholly dependent on the delicacy of the measuring apparatus, rather than on the qualities of the material tested.

Thus the first two definitions undertake to fix a *limit*, and evidently the position of this limit is simply the point where either the permanent set or the deviation from a linear relation between load and deformation becomes *measurable*. If one can measure accurately to 0.0001 inch, he will discover these limits earlier, or at a lower stress, than if he can only measure to 0.001 inch. The French Commission recommend that measurements be made to

the nearest 0.001 mm. or to $\frac{1}{25400}$ inch. To discover a permanent set, furthermore, requires a constant release of the load, which is liable to disturb the deformation-measuring apparatus, and in any case the load at which a permanent set occurs can only be said to lie between two particular loads, the greater of which has produced the first permanent set observed. The time and trouble involved in releasing the load so often will act to remove this test from nearly all scientific and commercial work.

It has generally been assumed that the first two definitions of elastic limit given in Art. 261 locate identical points in the stress-diagram, but

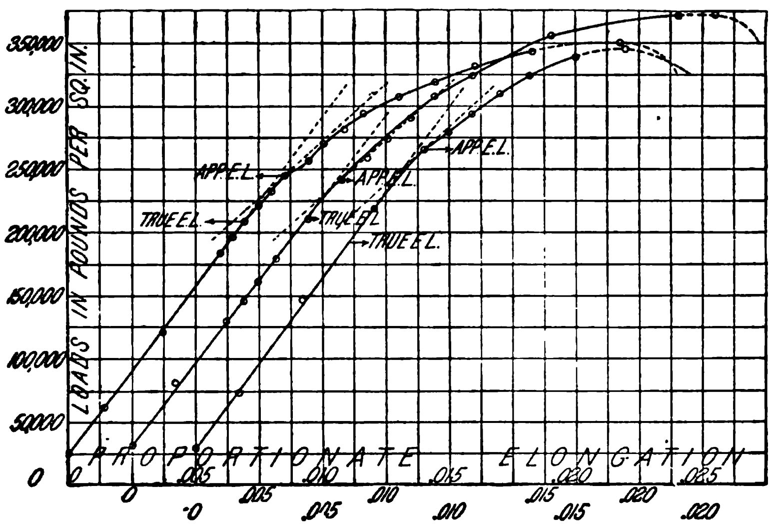


FIG. 249.—Tension Tests of Steel Piano-wire. Gauged length 6 in., Diam. 0.04 in. (*Wat. Ars. Rep.*, 1894.)

with the most delicate measuring appliances it is found that these two definitions may locate points very far apart.

The third definition is also indefinite, since it remains a question as to which load is to be taken, the higher load at which the first great permanent elongation occurred, or the lower load under which this elongation continues to spread throughout the entire length of the bar. These often differ as much as from 3000 to 6000 pounds per square inch. (See Figs. 7 and 8.)

Again, when the most delicate apparatus is employed, several specimens from the same bar of the most uniform material may give elastic limits of either of the first two kinds which differ widely from each other and hence become mutually contradictory. In other words, *such delicate tests are quite worthless for all practical purposes, i.e., the results are not characteristic.*

263. The Apparent Elastic Limit.—The term "relative elastic limit" was coined by the author in 1891, to be used in his work of testing timber for the Forestry Division of the U. S. Agricultural Department. He then

defined it as the point on the stress-diagram (of tests in cross-bending) where the rate of deformation is fifty per cent greater than it is at the origin (see Figs. 247, 248, and 249). To find it draw a tangent to the stress-diagram at the origin, and then lay a parallel ruler on a line making with the load line an angle whose tangent is fifty per cent greater than that of the original tangent line, and then move the ruler until its edge becomes tangent to the stress-diagram, and draw such tangent line. The "relative elastic limit" is then located by eye as this point of tangency. In the tests of wooden beams this point usually falls on the diagram where its curvature is about the most rapid (radius of curvature a minimum), and *in all tension stress-diagrams of the various metals it will be found to mark a well-defined point, whose coordinates are practically fixed and constant for the same material.* While this point is certainly beyond all "true elastic limits" when defined as *limits*, yet *in the metals it will be found to fall very little beyond such true limits.* In fact it commonly falls below the "elastic limit" recorded in all the Watertown Arsenal Tests, where the deformations were measured and recorded to the nearest $\frac{1}{10000}$ of an inch, as shown in Figs. 9 and 248, and many others in Chapter XXIV.

The French Commission make use of the term "apparent elastic limit" to indicate what in England is called the "yield-point" or "break-down point," and in Germany is called the "beginning of great elongations." But with such materials as cast iron, high carbon-steel, and often with wrought iron and other metals there is no "yield-point," or no point where the material deforms under a constant load, unless it be at the point of maximum load, to which of course it is not intended to apply. The term "apparent elastic limit" in this sense, therefore, has not a universal application, and hence cannot be used as the commercial elastic limit to be employed in practical tests.

The term "apparent elastic limit" has not as yet come into use in English; and if it now be defined arbitrarily as the term "relative elastic limit" is defined above, it could have this specific meaning and would be of *universal application.*

When so located it will be found at practically the same point on all tests of like kind on similar materials. It is therefore *characteristic of the material.*

It does not require the use of expensive and troublesome appliances for its accurate location. Relatively crude appliances can be used to measure the deformations; and though these may not fall on a smooth curve, a mean line drawn through them, as the stress-diagram, furnishes a satisfactory means of locating this "apparent elastic limit." It is therefore *a practically determinate function.*

It can readily be determined from an automatically registered stress-diagram, of any description, thus admitting of a continuous and uninterrupted progress of loading, so that the conditions of a test can be exactly duplicated as to speed, which cannot be done when the test is stopped to

take deformation readings. It is therefore *a true relative limit of the elastic field.*

Although this point is slightly beyond the true elastic limit, it will mark a point corresponding to a permanent set much less than can be measured on any scale by the naked eye,* and hence it may be regarded as the *true elastic limit for commercial purposes.*

It serves perfectly to classify materials as to the maximum loads they can resist without receiving deformation which would injure them for continued service, these being the real "ultimate loads" for all practical purposes. It marks, therefore, the most valuable and important property of all engineering or building material. In other words, *it is the most essential characteristic point on the stress-diagram.*

In the opinion of the author of this work, therefore, an "apparent elastic limit," defined and determined as here described, is the best if not the only satisfactory solution of this troublesome question. The fifty per cent increase in the rate of deformation was chosen as being about the least which would mark a well-defined point of tangency on the stress-diagram. Since it always marks a point which corresponds to an extremely small permanent set, there would seem to be no objection to its use.

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* Seldom more than one one-thousandth of one per cent of the measured length. It is a maximum in the case of the high-grade, hardened steel wire, shown in Fig. 249, where it reaches about $\frac{1}{50}$ of one per cent.

CHAPTER XV.

TENSION TESTS.

264. Significance of Tension Tests.—Tension tests are at once more common, more readily made, and more useful in revealing the true character of a metal than any other kind of mechanical test. In fact, when other kinds of tests are made it would commonly be well to accompany them with a few tensile tests for the purpose of being able better to coordinate the results with those obtained on other materials by similar tests, or on like materials by different tests. In this connection, however, it is well to remember that all the metals are wanting in strict homogeneity, and that they may be regarded as aggregations of more or less dissimilar elements embedded in a common matrix, somewhat like granite. (See Arts. 105 and 108.) For instance, the planes of rupture will be different for different kinds of tests on the same specimen, and hence the strength developed will be that of a different combination of elements in each case. Also, the strength to resist various kinds of stress may lie in entirely different elements of the aggregation, as, for instance, in the case of cast iron the strength to resist tension is the strength of the graphitic carbon matrix in which the iron crystals are embedded, while the strength in compression is largely the strength of the iron crystals themselves.*

What we call the maximum strength of the material, therefore, or its strength at rupture, is not usually the sum of the maximum resistances of the several elementary portions of the cross-section, since they do not all distort equally. It is often the case that actual rupture occurs successively over many elementary portions of the broken section before the final failure occurs. More especially is this true of the elastic limits of the material, while with iron and steel castings this failure in detail is so prominent as to cause the stress-diagram to be a curve almost from the beginning of the loading. Here, too, the irregular shrinkage often leaves very great internal stresses in the body, which causes some portions to come to their elastic limits and ultimate strength much earlier than others, again giving rise to a curved stress-diagram.

For these reasons we find, when the most delicate means are employed to measure deformations under increasing loads, that in almost no case is the

* M. Osmond.

deformation strictly proportional to the load, and that even very small loads will produce some little permanent deformation or set. This is why the definitions given in Art. 261 must depend on certain arbitrary limits of deformation and set, and are not true absolutely as they have hitherto commonly been defined.

The tension test is especially well calculated to show what local irregularities may be found in a finished product, and to indicate to what extent the work of forging (rolling or hammering) has produced that degree of homogenity expected of it.

The tension test is more readily standardized than any other so as to be independent of "personal equation" and of variations in the testing-machines employed. It also demands the least amount of preparation of the test specimen, if tests are to be made only for commercial purposes. Except for the inherent want of uniformity or of homogeneity mentioned above, therefore, the tension test may be made to give typical and uniform results, and it should be considered as the best single test to make on any of the metals.

265. Selection of the Test Specimens.—Test specimens may be taken either from the finished product or from the material when poured if it is derived from a fluid condition. In American steel mills it is common to roll (or hammer) a test rod from a small ingot poured from each heat, whether it be of the Bessemer or the open-hearth process, and the maker depends on the tests made on this bar to guide him in the further use of the ingots poured from that heat. The user also is often satisfied with these tests, especially when his requirements are not very rigid.

In making iron and steel castings it is common to have test samples cast in the same moulds with important castings, and joined thereto, so that they shall represent, of necessity, the identical metal of which the structural form is composed. If special moulds are used for these test specimens, they should be of dry sand, under a head of at least eight inches, and with an inclination of at least one in five to allow the escape of the gases. With very heavy castings (over three inches in thickness) test specimens may be cut from the head itself which is cut off from the upper end of the casting.

If the specimens are taken from rolled structural forms, they should be taken from the thicker parts, which have received the least work in the rolls. The thinner parts are always harder, have higher elastic limits and greater ultimate strength.

With wrought iron a great difference will be found in specimens cut with and across the direction of the rolling, the former having much higher strength and a greater ductility. In steel plates there is little difference, and in rolled brass and copper plates there is no difference. In the case of the bronzes it is necessary to have test samples poured from different parts of the same melting, as the mixture changes its characteristics rapidly when in a melted state.

266. The Preparation of the Test Specimen.—In order that the test specimen may fairly represent the material under examination, or the par-

ticular plate, or bar, or rolled form from which it is to be taken, it is necessary to observe a number of rigid requirements.

The specimen must be obtained by cutting it out in a way that will leave it perfectly straight. If it is bent in getting it out, it should be heated to straighten it; but this may often change the original molecular arrangement, and should be avoided if possible. When the specimen is cut from a larger portion of a plate or rolled form by shearing, it will invariably take a curved form. In this case *the plate, or form, should be sheared away from the specimen*, in narrow slices, so as to leave the test specimen unbent. If the specimen is bent and then straightened, it raises the elastic limit and hardens the metal, the same as any other kind of cold working. Instead of shearing, some milder process, such as planing or drilling or sawing, should be resorted to to obtain the test specimen. For, besides the bending action on the bar as a whole, the effect of the shearing or punching is to seriously injure the metal for about an eighth of an inch beyond the sheared surface, leaving it so non-ductile, or brittle, that it will not elongate appreciably, and hence under a tensile test these surfaces will be severed very early in the test, and the cracks so started may cause the remainder of the cross-section to tear asunder in detail. To prevent this action on sheared or punched specimens, at least an eighth of an inch of thickness should be removed from all punched or sheared faces, by reaming, planing, or filing. The effect of not doing this is shown in various figures in Chapter XXVI, where both punched and drilled test specimens of one-fourth inch iron and steel plates had been grooved for testing, leaving varying widths of metal between the bottoms of the grooves. The effect of this variation in width is also here shown. Various bending tests there shown also exhibit the weakness resulting from shearing and punching.

The ordinary lathe, planer, and milling-machine tools are not suitable for the *final finishing* of the specimen, as they tear and bruise the remaining metal, giving rise to a condition favorable to the starting of incipient fractures at the surface of the specimen. These tools may be used for roughing out the shape desired, but *it should be finished with the file*, and in case of the softer metals, like copper, the file should be followed with fine emery-paper.

Castings should be cut down about an eighth or a tenth of an inch from the rough exterior, on the reduced section, and they should also be trued-up on the ends which are to be gripped.* Rectangular edges should always be taken off by a file to remove any incipient cracks or irregularities which may be left here by a kind of crushing-down action of the tools operating on one or both of the plane faces meeting on these lines. Test specimens of the softer metals should never be beaten with a steel hammer, but with wooden or copper mallets, if it is necessary to use such means to straighten them.

Standard shapes of test specimens are shown in Fig. 251.

* For commercial purposes a cheap form of specimen which is tested without turning down at all is shown in Chapter XXIV.

RULES OF THE FRENCH COMMISSION FOR TENSION TESTS.

1. The test should be continuously progressive.
2. The duration of the test should in a general way increase with the volume of the specimen.
3. For standard tests on specimens of ordinary dimensions of which the sectional area is not more than one square inch (600 sq. mm.) and the measured length not more than eight inches (20 cm.) it seems that the duration of the test should be included between one and six minutes.
4. For test specimens having a thickness less than 0.2 in. (5 mm.) the duration of the test should be less than thirty seconds.
5. It is necessary to avoid, especially with soft metals, producing a sensible heating of the test bars.

267. Standard Dimensions of Tension-test Specimens. — It has been shown * that so long as the test specimens of a given material maintain the

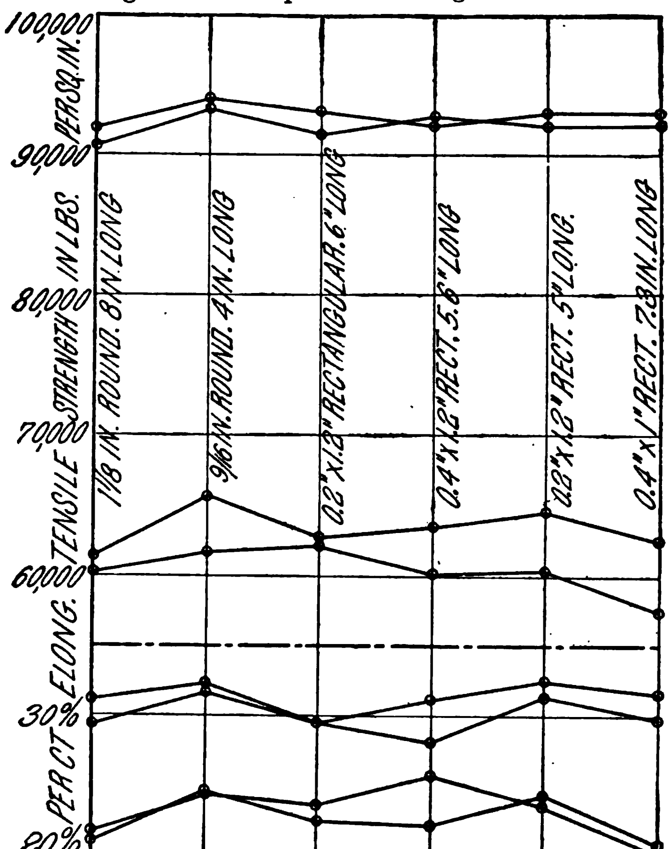


FIG. 250.—Showing the Constancy of the Strength and of the Elongation when $\frac{l}{\sqrt{A}}$ = a constant (8 in this case). Each result is the mean of five tests on the same material. (*French Com. Rep.*, 1894, vol. III. p. 72.)

* By MM. Lebasteur, Marié, and Barba. See *Rep. French Commission*, vol. III. . 23.

same relative dimensions, or are geometrically *similar* in form, the strength and the percentage of elongation remain constant, as shown in Fig. 250. The French Commission have, therefore, adopted the relation $l^2 = 66.67A$, or for cylindrical specimens $l = 7.2d$, where l is the measured length on which percentage of elongation is computed. An eight-inch specimen would then be 1.11 inch in diameter, or nearly 1 sq. in. in area of cross-section. Since this relation between l and A was chosen for convenience (for $l = 200$ mm., $A = 600$ sq. mm.), persons using inch units might well choose the relation $l = 8d$, or

$$l^2 = 81A. \qquad (1)$$

For square sections, therefore,

$$l = 9b, \qquad (2)$$

while for round sections

$$l = 8d, \qquad (3)$$

these being intermediate between the French and the German standard dimensions.*

Equations (1), (2), and (3), therefore, may be employed in finding what length of specimen to use to give comparable and consistent percentages of elongation, when the excessive elongation near the broken section is included.

If it is practicable to so prepare the specimen as to make the area of the cross-section nearly constant, then a fixed length of specimen could be used for all tests. Thus for the standard length of eight inches the diameter of round specimens would be one inch. Where the cross-section varies from this the lengths should vary in accordance with equation (3). Thus

For diameters of $\frac{7}{8}$ inch make $l = 7$ inches.
" " " $\frac{3}{4}$ " " $l = 6$ "
" " " $\frac{5}{8}$ " " $l = 5$ "
" " " $\frac{1}{2}$ " " $l = 4$ "
" " " $\frac{3}{8}$ " " $l = 3$ "
" " " $\frac{1}{4}$ " " $l = 2$ "
" " " $\frac{1}{8}$ " " $l = 1$ "

For plate tests we have, from (2),

$$l^2 = 81A = 81bt, \quad \text{or} \quad l = 9\sqrt{bt}. \qquad (4)$$

Since it is common to prepare several of these together in a milling-machine, it is desirable to have a common width for these tests, and a width of one inch has been usually employed in America. This may be done if the lengths are varied to give comparable results. Thus, from eq. (4), $l = 9\sqrt{bt}$, the following scheme of lengths and thicknesses is derived, the width being one inch in all cases:

* The German Commissions have agreed on $l = 11.3\sqrt{A} = 10d$ for round section and $11.3b$ for square sections.

For plates $\frac{1}{4}$ inch thick make $l = 4\frac{1}{2}$ inches.
" " $\frac{3}{8}$ " " " $l = 5\frac{1}{2}$ "
" " $\frac{1}{2}$ " " " $l = 6\frac{1}{4}$ "
" " $\frac{5}{8}$ " " " $l = 7$ "
" " $\frac{3}{4}$ " " " $l = 7\frac{3}{4}$ "
" " $\frac{7}{8}$ " " " $l = 8\frac{1}{2}$ "

For thin sheet metal make the measured length always four inches, and use three standard widths as follows:

For thicknesses from 0.1 inch to 0.2 inch make width $= \frac{1}{2}$ inch.
" " " 0.05 " " 0.1 " " " $= \frac{3}{8}$ "
" " less than 0.05 inch " " $= \frac{1}{4}$ "

All these relative dimensions agree closely with those recommended by the French Commission (1895).

The measured portion of the reduced section (called l in the above discussion) must be removed from the shoulders by a distance at least as

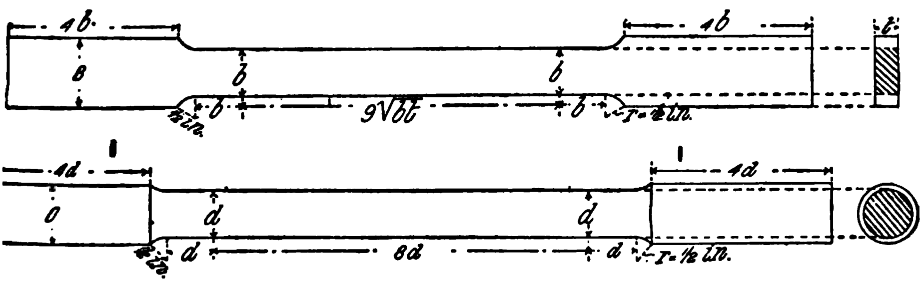


FIG. 251.—Standard Dimensions for Rectangular and Cylindrical Test Specimens.

great as the diameter or thickness of the test bar, in order to avoid the effect of these enlarged portions in reducing the elongation. The reduction of the percentage of elongation near the ends is well shown in Fig. 252, where the bar had a total reduced length of 15 inches and a diameter of $1\frac{1}{8}$ inches. Steel bars of this shape will break near the centre, while wrought-iron bars will break at various distances from the centre, with a more uniform distribution of the elongation.

268. Tetmajer's Analysis of the Elongation of Tension-test Specimens.*— The typical forms of tension-test specimens are shown in Fig. 251, where both round and rectangular sections are given. It has been shown by numerous experiments that the strength and the reduction of area are somewhat dependent on the form of the test specimen, while the elongation is very greatly dependent on these relative dimensions. This is well illustrated in the reproduced photographs shown in Fig. 10, and also in the elongation-diagrams Figs. 252 and 253. Here are shown first the original specimens, then the specimen stretched to its maximum loading, the elongation being nearly uniformly distributed over the length, and finally the specimen greatly reduced at one point where rupture is about to occur. It is evident,

* Tetmajer's Communications, vol. IV.

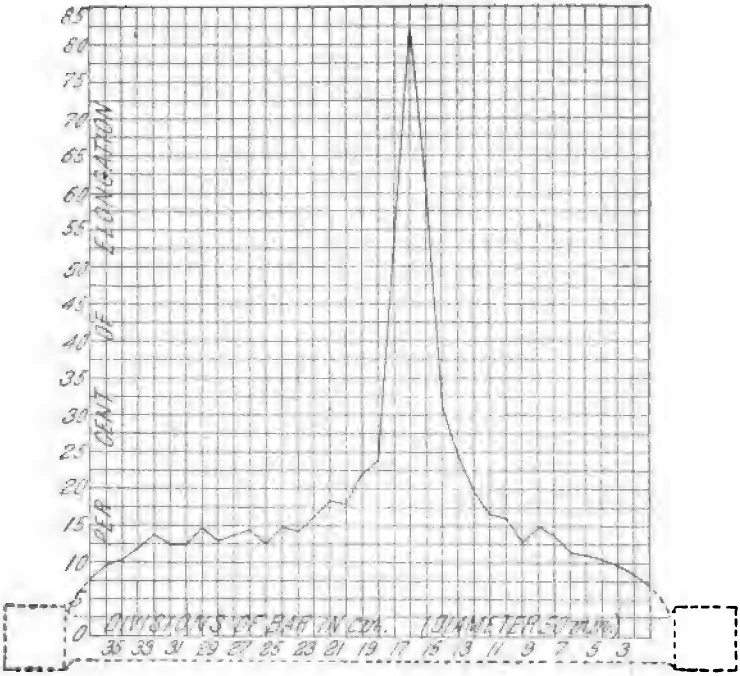


FIG. 252.—Showing Distribution of the Elongation over a Steel Bar 15 in. Long and 2 in. in Diameter. (*Rep. Fr. Com.*, vol. III, Pl. II.)

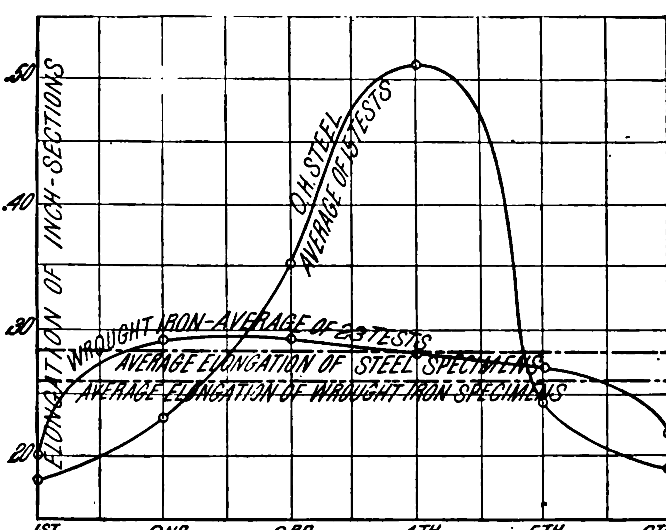


FIG. 253.—Showing the Variation in the Distribution of the Elongation of the Several Inch-spaces of Six-inch Test Bars of Steel and Wrought Iron 0.56 in. in Diameter. (*Wat. Ars. Rep.*, 1890.)

from a study of these specimens, that the total elongation of tension-test specimen may be divided into two distinct parts, namely:

1. The general elongation.
2. The local elongation.

It will further appear that the local elongation is nearly the same in all cases, and is practically independent of the length, while the general elongation, having occurred uniformly along the bar, is directly proportional to the length.

If $l =$ measured length of specimen,

$\Delta l =$ total elongation,

$\lambda =$ proportional distributed elongation $= \dfrac{\text{distributed elongation}}{l}$,

$\Delta l_e =$ total local elongation

then λ may be found for any given test by measuring Δl for two lengths of the same specimen, each to include the broken section. Since the standard length of specimen is 8 inches (200 mm.), it usually will be found convenient to use 8 inches and 4 inches for these two lengths. If the specimen be marked originally every inch, then after it has broken two sets of these marks may be chosen, each pair to include the fracture, and to be points originally 8 inches and 4 inches apart respectively. Then we may have

$$\Delta l_8 = 8\lambda + \Delta l_e$$
$$\Delta l_4 = 4\lambda + \Delta l_e$$
$$\Delta l_8 - \Delta l_4 = 4\lambda$$

or

$$\lambda = \tfrac{1}{4}(\Delta l_8 - \Delta l_4). \qquad (1)$$

This function, λ, *obtained in this way, is independent of the length of the specimen, and is the true characteristic elongation of the material.*

It would seem that this function is the one to be generally adopted as the true index of the ductility of the material. Unfortunately custom has established $\dfrac{\Delta l}{l}$ as

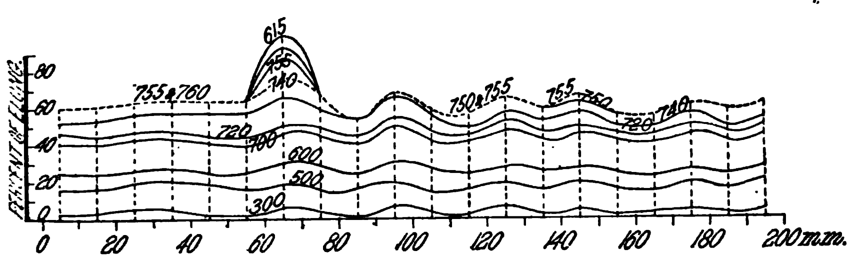


FIG. 254.—Elongation of a Specimen of Copper 200 mm. Long for the Loads as given. (*Fr. Com. Rep.*, vol. III. Pl. VII.)

the ductility function, or "the percentage of elongation," and this varies greatly with the ratio of length to form and area of cross-section.

Professor Tetmajer shows that the following relative dimensions give practically equal percentages of total elongation:

(A) CYLINDRICAL SPECIMENS.

Diameters	0.4 in.	0.6 in.	0.8 in.	1.0 in.
Observed length	4.0 in.	6.4 in.	8.0 in.	10.0 in.
Mean observed elongation, "Phœnix" steel	30.1%	30.4%	30.5%	30.6%

(B) RECTANGULAR SPECIMENS, ALL 0.4 IN. THICK.

Ratio : $\frac{b}{t}$ =	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
Observed length, inches..	4.0	4.8	6.0	7.2	8.0	8.0	8.0	8.4	9.2
Mean observed percentage of elongation..........	27.0	27.2	27.2	26.8	26.1	25.7	26.1	26.7	27.0

Each of the above observed elongations in Table (A) is the mean of five tests and in Table (B) of ten tests, and the results indicate that equivalent lengths were used. When the length was taken as 8 inches in each case, the percentage of elongation in Table (A) ranged from 26.5 to 32.4, while in Table (B) it ranged from 21.3 to 28.6. These results are consistent with the rules laid down by the French Commission and which the author has interpreted approximately in English measures in the previous article.

269. The Time Function of Tension Tests is not an important one. Bauschinger has shown that within the ranges of practicability the time element is of no consequence. This is also shown in Fig. 255, where results

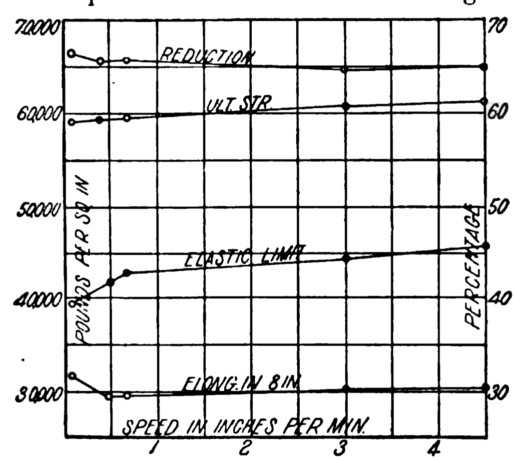


FIG. 255.—Showing the Effect of Pulling Speed of the Testing-machine on the Recorded Results with Structural Steel. (Campbell's *Structural Steel*, p. 253.)

obtained with the greatest rapidity obtainable in the American standard testing-machines are compared with results from very slow tests, with very little difference in the results. The greatest recorded differences are found in the elastic limit; but as these were all observed by the "drop of the beam," it is likely that the speed had more to do with the obtaining of the reading than it had with the real action of the material itself.

270. Tension-test Machines.—There are three general types of universal (tension, compression, and cross-bending) testing-machines on the American market, viz., hydraulic machines, screw-gear machines, and the Emery machines. The hydraulic-power machines have, however, been practically abandoned in favor of the screw-gear for all experimental and commercial

purposes, while for high scientific accuracy and an incredible delicacy nothing ever made can compare with the Emery machines.* The first two varieties of machine owe their present high state of development largely to Mr. Tinius Olsen, and the Emery machine was originally designed by Mr.

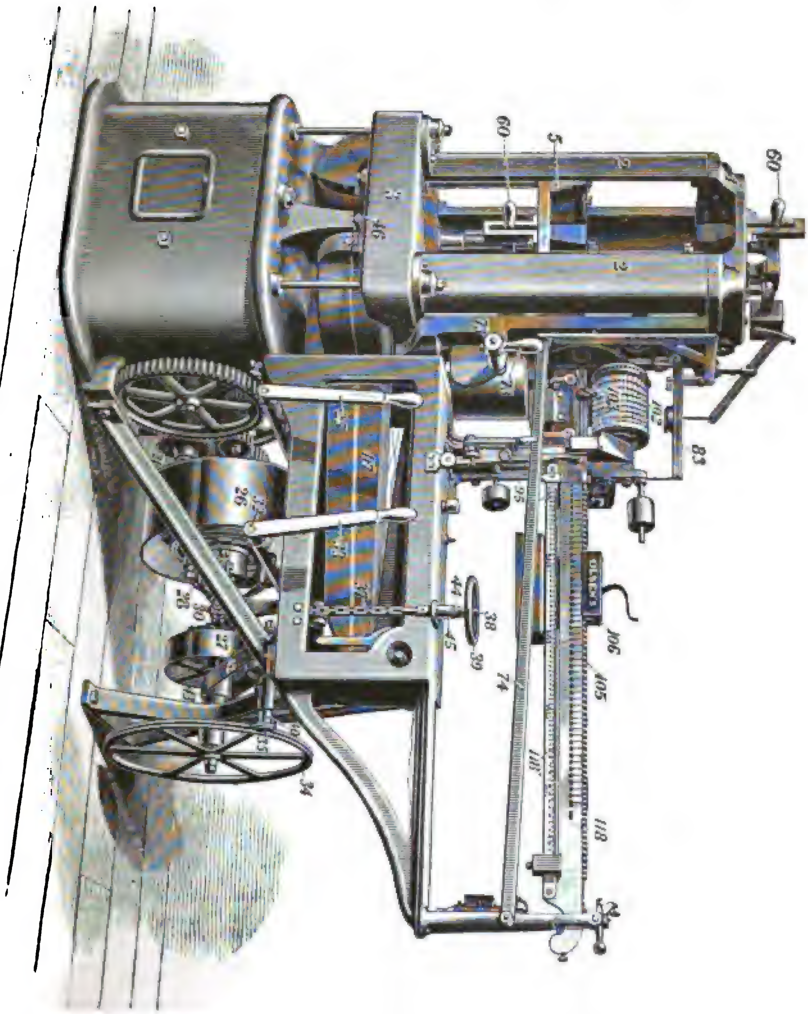


FIG. 256.—Olsen Screw-gear Testing-machine.

A. H. Emery, but has since been greatly simplified and improved by Wm. Sellers & Co., the present manufacturers.

The hydraulic machines have some advantages when operated by hand, but they all have the disadvantage that it is impossible to maintain a given

* The hydraulic and screw-gear machines are made by Tinius Olsen and by Riehlé Bros., and the Emery machines by Wm. Sellers & Co., all of Philadelphia, Pa.

load without continuous pumping to supply the small leakage which always occurs.

The Olsen screw-gear 100,000-lb. testing-machine, shown in Fig. 256, will be taken as a type of the testing-machines which are almost exclusively employed in America. The power is applied to the pulleys 26 and 27, the former used for direct (downward) and the latter for reverse (upward) motion of the moving cross-head, 5. Extremely slow speed is obtained by throwing into contact the small friction-gear 35, operating upon the large wheel 34, which is rigidly attached to the driving-shaft. This is effected by drawing on the chain 37 by turning the hand-wheel 39, which tightens the band 42 and starts 35 to revolving. The band-wheels 26, 27, and 43 all revolve freely on the driving-shaft, except as 26 or 27 is made fast to it by the friction-clutch 28, 30, through the hand-lever 33.

A medium speed of the moving cross-head is obtained by simply throwing 26 in gear by the hand-lever 33, and a high (upward or downward) speed is attained by changing the gears by means of lever 25, the upward speed greatly exceeding the downward because of the different band connections on the direct and on the reversing-pulleys 26 and 27 respectively.

The moving cross-head, 5, is brought down by the turning of four screws, one at each corner, only two of which are visible in Fig. 256. A tension-test specimen is placed between this moving head-piece and the fixed cross-head above, being gripped in each by means of hardened steel wedges with grooved faces. The pull on the specimen is thus transmitted through the four cast-iron columns, 2, to the weighing-table, 3, which rests by means of fixed spurs upon the three weighing-levers, 117. Between these and the weighing-beam, 118, there is one intermediate multiplying-lever (not numbered). The large poise, 106, in this machine is supposed to be moved by means of the screw 105, which is under automatic electric control. When the beam lifts, the screw is put in motion; and when it leaves its upper contact the screw stops its motion. The weighing-beam is thus automatically maintained in constant balance. If operated by hand, the large poise, 106, is set forward by full revolutions of the screw by means of the handle shown, and the intermediate loads indicated by balancing with the small poise shown at the right-hand end of its scale, 118′. When operated automatically this poise is not used, and the fractional part of the total load is read on a graduated disk attached to the screw at the left-hand end, but not clearly shown in the figure.

Compression tests are made by attaching a compression-block to the lower side of the moving cross-head, and inserting the specimen between it and the weighing-table, 3.

Cross-breaking tests are made by placing the end bearings on the weighing-table (or on an I-beam resting on this table if the specimen is long), and attaching a knife-edge bearing to the lower side of the moving cross-head.

A machine in nearly all respects quite similar to the above is that shown in Fig. 257, made by Riehlé Bros. The poise here is moved by a chain

passing over a driving-pulley, which pulley is operated either by hand or by power under electrical control, the same as the screw in the Olsen machine. Only two screws are here used for moving the pulling cross-head, instead of four as in the former case. Both forms of machines are made in the highest style of the art, both being the survival of the fittest in a long succession of types of testing-machines. They are by far the most useful and convenient testing-machines made,* and are not likely to undergo much change in the

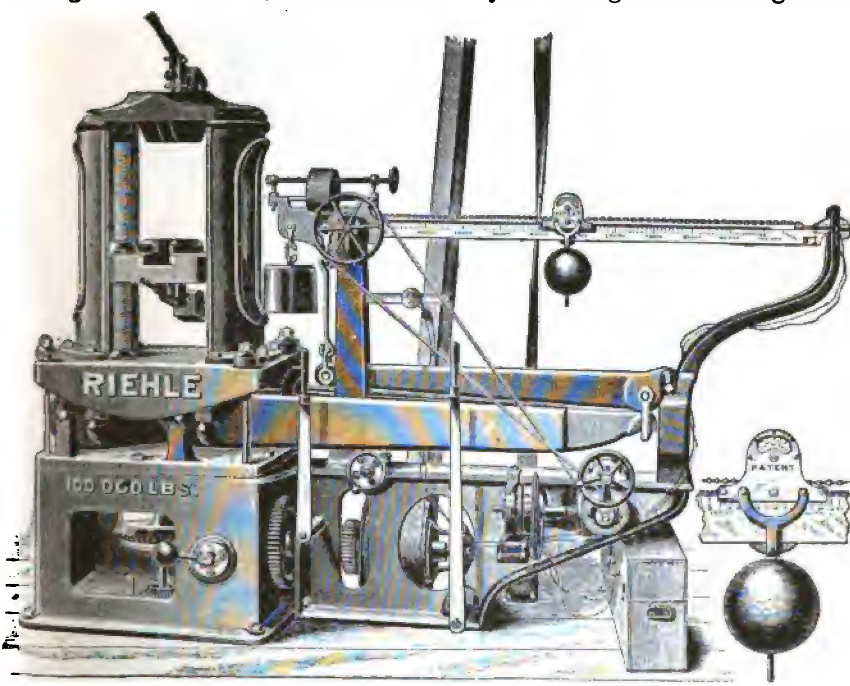


FIG 257.

future. (They are now, 1896, being sold in Europe.) The speeds at which the machine shown in Fig. 257 may be driven directly are as follows: $\frac{1}{10}$ in. per min.; $\frac{1}{4}$ in. per min.; $\frac{3}{4}$ in. per min.; $1\frac{1}{2}$ in. per min.; and 8 in. per min. For tests of low ultimate strength, speeds of $\frac{1}{2}$ in. per min. and of 4 in. per min. can also be used. The higher speeds, down to $\frac{3}{4}$ in. per min., can also be used in raising the moving head. By changing the speed of the main shaft from which power is obtained all these speeds can be increased or diminished at pleasure. The speeds as given above are for 150 revolutions per minute of the driving-pulleys on the testing-machine.

In Fig. 258 is shown a small screw-gear power machine, made by Riehlé

* The author offers no apology for not giving descriptions of any of the scores of styles of machines which have been built and which are still in use—mostly in Europe. They will never be built, bought, or used in this country, and most of them can be found illustrated in the Report of the French Commission, 1895, vols. II. and III.

Bros. in capacities of 20,000 lbs., 30,000 lbs., 40,000 lbs., 50,000 lbs., and 60,000 lbs., with hand-power attachments, and automatic weighing appliances if desired. With the 20,000-lb. machine the hand-power does very well,* although steam or electric power is always preferable. Autographic

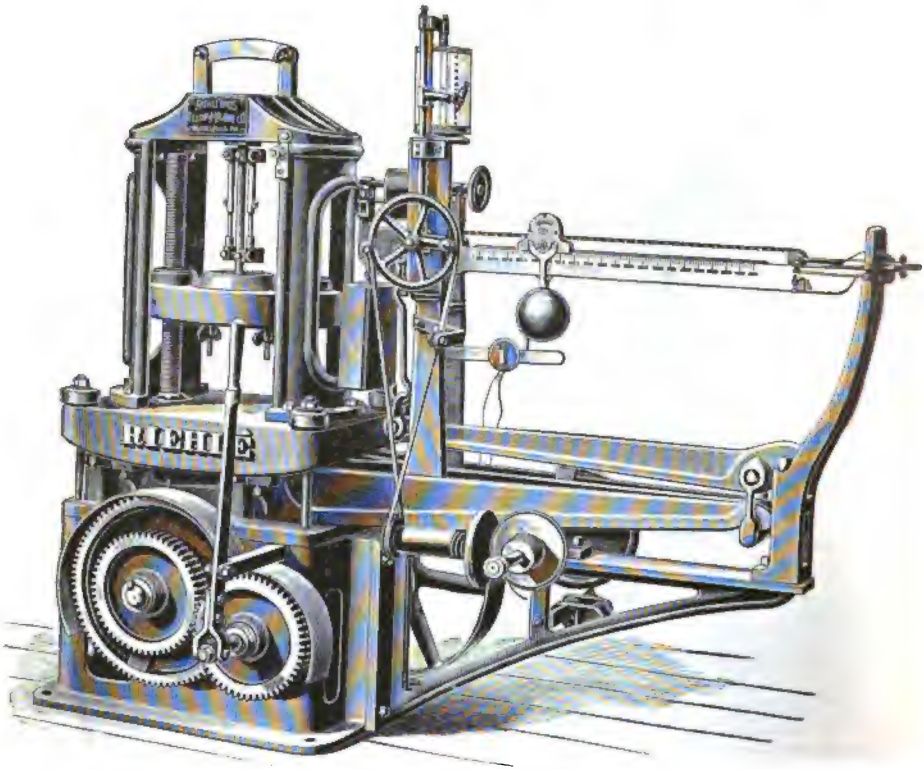


Fig. 258.

recording attachments (see Art. 275) are attached to these the same as with the larger machines. They are too small, however, for general commercial purposes.

271. Gripping Devices.—A great variety of gripping devices have been employed, such as eyes and pins, shoulders and split sleeves, screws-threads and nuts, and plain bars with wedge-grips This last form has now replaced all others in America except such as may still be employed on some of the older machines. For round specimens notched grips are used, while with square or flat specimens the plain wedges are employed. The Riehlé plain grips are swelled in the centre so as to grip the specimen hardest along its

* These small machines are the best patterns for students' use. It is better to have several of these than one larger machine. They serve almost every purpose in a course of study on the strength of materials.

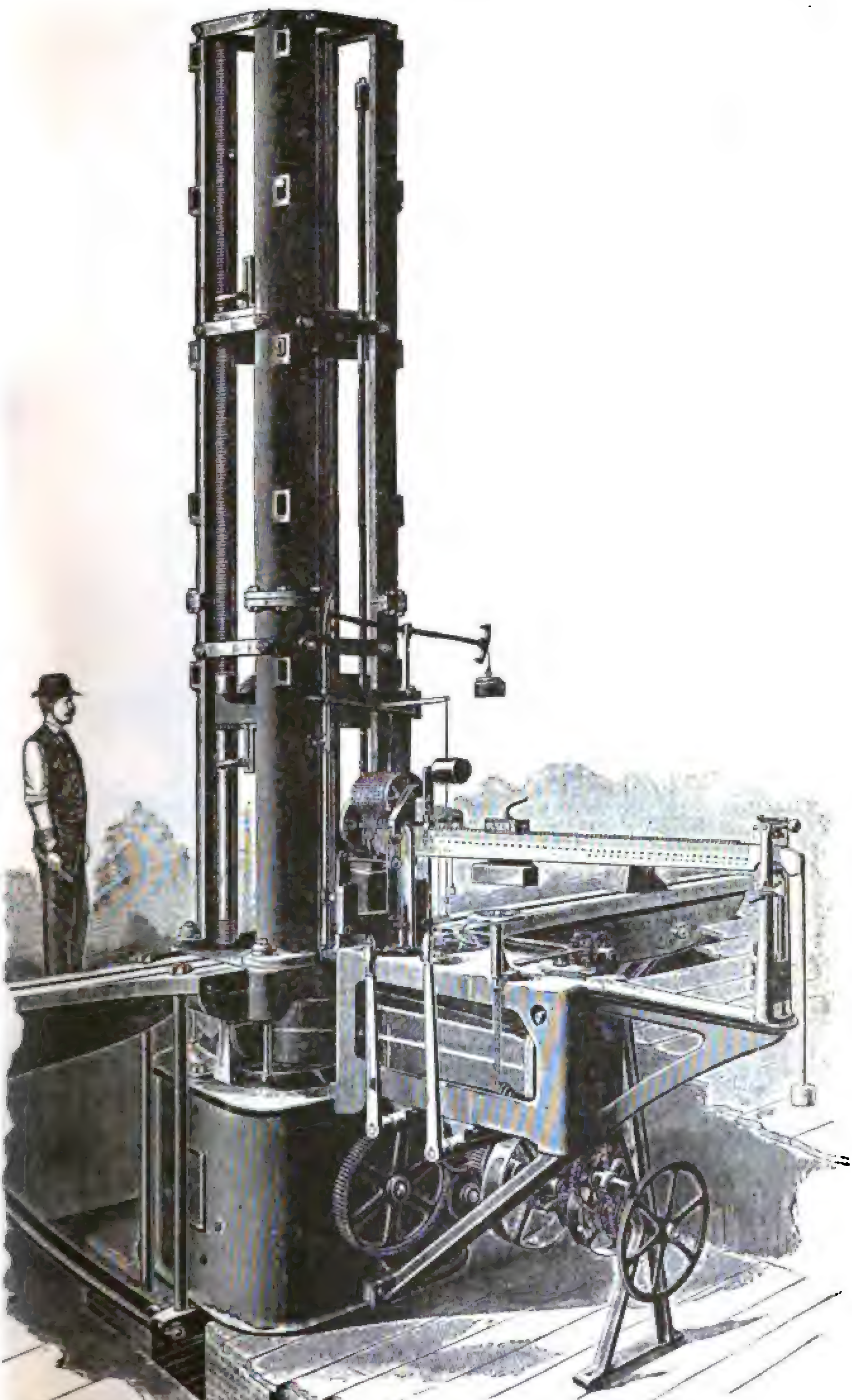


FIG. 259.

axis of symmetry. The Olsen grips are swivelled on spherical bearings at the back to enable them to more readily adjust themselves to the specimen.

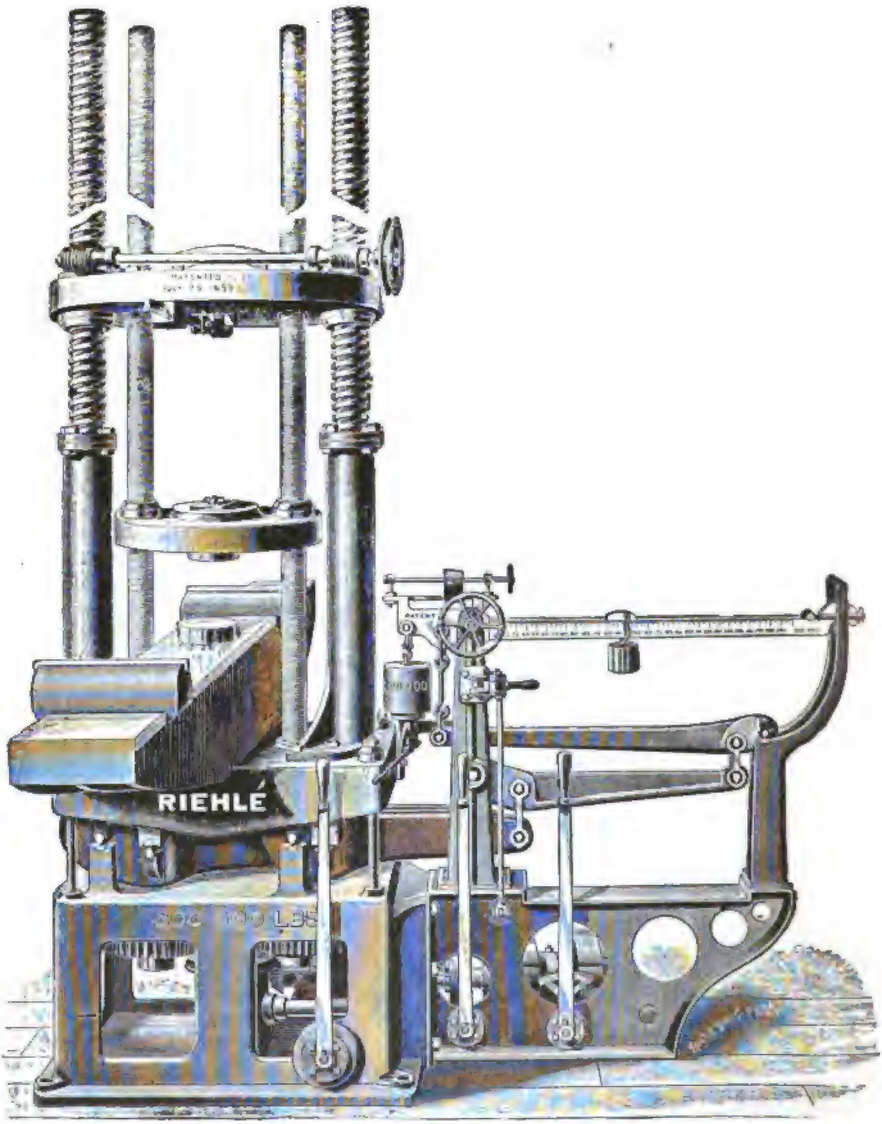


FIG. 260.

The wedge-grips can be used with plain bars and plates, without any reduction of section, as well as with specimens specially prepared by turning down or by a milling-machine.

272. Similar Machines for Various Special Purposes.—In Fig. 259 is shown an Olsen machine designed for testing *full-sized structural specimens*, whether tension or compression members, or beams, and made in capacities of 200,000, 300,000, and 400,000 lbs. These machines have all the rates of motion, and the automatic weighing and autographic recording appliances described with the smaller machines. The height may be made almost anything which may be desired.

In Fig. 260 is shown a similar machine of 300,000 lbs. capacity, in which the four cast-iron corner-posts are replaced by two large compression-screws.

Fig. 261 illustrates a convenient and cheap machine for testing hoop-iron

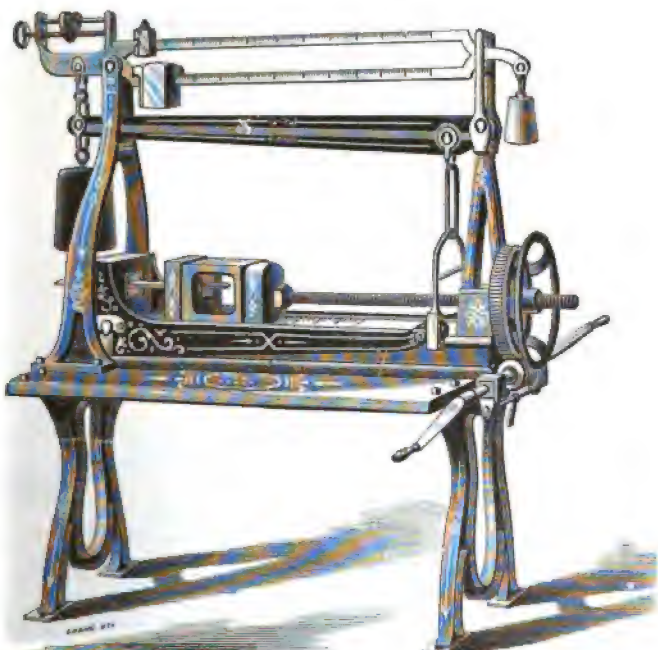


FIG. 261.

and round bars, in tension only, of 20,000 lbs. capacity, suitable for office use.*

In Fig. 262 is shown one form of a *wire-testing machine*, made by Olsen, in capacities of 10,000, 15,000, and 20,000 lbs. It is also adapted for testing band-iron and other forms, and could also be used for compression and cross-breaking. (See also other wire-testing machines in Chap. XXXIII.)

Fig. 263 shows a form of cloth- and paper-testing machine,† having a

* Made by Riehlé.

† Made by both Olsen and Riehlé.

capacity of 200 lbs. on one inch in width of material, while Fig. 264 shows another form, for paper only, of 100 lbs. capacity. The stress is indicated on the face of the dial.

Tension-testing machines for breaking cement briquettes are shown and described in Art. 324.

273. The Emery Testing-machine.—This is without doubt by far the most perfect weighing-machine ever devised. It was originally invented

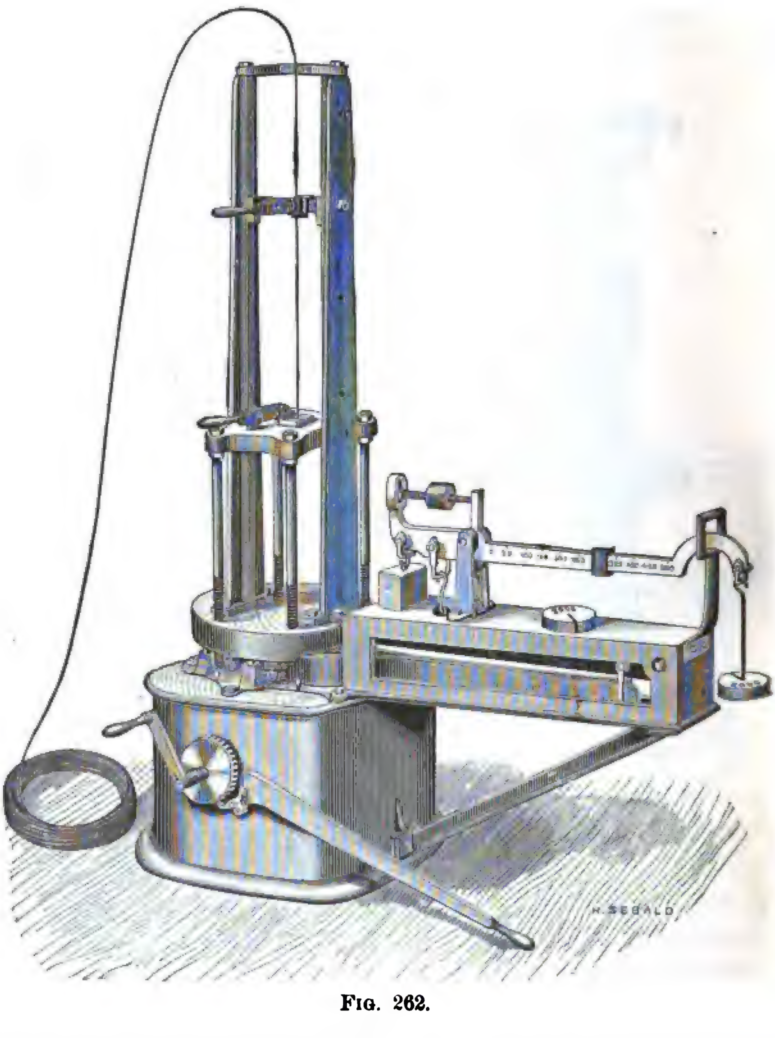


Fig. 262.

and constructed by Mr. A. H. Emery, C.E., for the use of the U. S. Test Board in 1879, and this first machine, having a capacity of 800,000 lbs. in both tension and compression, is still in daily use at the U. S. Arsenal

at Watertown, Mass. It is now manufactured in various sizes by Wm. Sellers & Co. of Philadelphia, who have modified and improved the original plans in many respects. It is not too much to say of this machine that it operates absolutely without frictional resistance, tests with equal accuracy large and small specimens, is easily and quickly operated, and is practically indestructible by any amount of legitimate use. The marvellous character of this machine merits a careful study

FIG. 263.

FIG. 264.

by all students in engineering, and an attempt is here made to adequately describe it.*

The essential principle of this machine consists in a means of transmitting a definite percentage of the force applied to the specimen to the scale-beams, and there weighing it accurately, without any friction whatever in the receiving, transmitting, or weighing parts. Hence any very small increment of the force applied is weighed with equal accuracy, whether this increment is added to a great or to a small previous load.† This is accom-

* The author has been greatly aided in this by drawings and descriptions made by Mr. Carl G. Barth, M.E., who is one of the joint inventors and patentees of the various improvements made on it by Wm. Sellers & Co.

† When the first machine was tested, a steel bar, 5 in. in diameter, was first broken under a load of 722,800 lbs., and then a single horse-hair was tested, and the machine gave the strength (16 oz.) of this as accurately as a small spring-balance which was used for a check. *Rep. U. S. Test Board*, vol. II. p. 1.

plished by means of two connected metallic sacks, or bags, of different sizes, the larger one, called the *hydraulic support*, receiving the full force transmitted through the specimen, while the other is rigidly held upon the

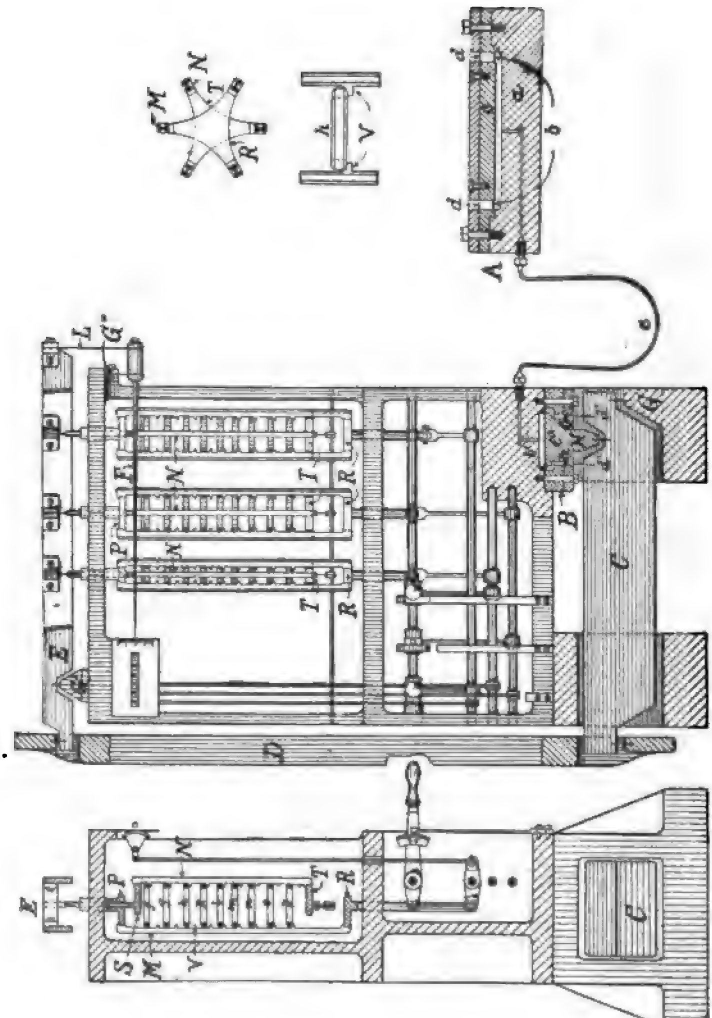


FIG. 265.—Weighing Apparatus of the Emery Testing-machine.

primary weighing-beam. Thus in Fig. 265, which is merely a schematic drawing, the load is received on the hydraulic support shown at A, and the pressure is transmitted by means of the enclosed liquid through the pipe e to the smaller sack at B, which being rigidly supported by the heavy cast-iron frame, G, shown in section above and below, the bearing-plate c' is forced downward, causing the block H to press upon the primary lever C.

This is then transmitted through *D* to the weighing-beam *E* and the indicator-arm *F*.

In place of knife-edges on the weighing-beams, thin plates of steel are employed, these being rigidly fastened in the two attached beams. These are so proportioned as to bring the combined bending and compression stresses produced in them well within their elastic limits. While these offer absolutely no friction, they do offer some elastic resistance, but this is all allowed for in calibrating or standardizing the machines. The weighing-beam is kept in balance by lowering upon it various weights which are placed upon the several poise-frames which are suspended from this weighing-beam. The particular manner of doing this, though ingenious and peculiar to this machine, is not essential and will not be further described here.* This part of the apparatus is entirely closed by a glass front, and it is never necessary to open it, the weights being imposed and removed from outside, and the load continuously indicated on a counter-cylinder shown in Fig. 265 at the left end of the indicator-arm *F*.

Passing now to a study of the latest form of the machine itself and its essential details, we have in Fig. 266 a view of the 200,000-lb. testing-machine made for Sibley College of Cornell University. To the left is seen the fixed *weighing-head*, containing the *hydraulic support*, and to the right the movable *straining-head*,† or hydraulic cylinder, by means of which the load is applied to the specimen and its deformation taken up. Both of these are supported and kept in alignment on a substantial wrought-iron girder-bed. In the background is also clearly seen the wooden case containing the scale, with some of the levers and poise-frames visible through its glass front, and also part of the hydraulic pump which supplies the straining-cylinder at either end, according as the machine is being used for a tension or a compression test. The supply- and exhaust-pipes are seen coming up through the floor at the end of the bed, and connecting with the straining-head by means of jointed pipes as shown. Rigidly secured to the weighing-head by means of nuts at each end of long bearings, one on each side, are seen the two horizontal *reaction-bars*, on which are cut continuous screw-threads. These form the rigid connection between the fixed weighing-head and the movable straining-head, through which they pass in smooth bearings placed sufficiently far apart to allow room—with several inches of clearance-space—for two large *abutment-nuts*, one on each screw. These nuts may be revolved simultaneously, by suitable mechanism, to produce motion for adjustment of the straining-head in either direction for different lengths of specimen.

The cut shows the machine ready for a compression test, the two *compression-platforms*, the one on the end of the *draw-bar* of the weighing-head,

* For a paper by J. Sellers Bancroft, on this machine, with full illustrations, see *Engr. News* and *Am. Machinist*, both of March 22, 1894.

† In this term the word "strain" is used as meaning deformation under stress. Thus the elongation (or compression) is all produced by the moving of this head to the right or left, thus producing the straining, or deforming, of the specimen.

the other on the end of the *straining-bar*, or piston-rod of the straining-head, also being plainly visible. With a compression specimen between the two heads the tendency is to move these apart, and the reaction-screws will then be in tension, requiring the abutment-nuts to be—as shown in the cut—up against the front bearings of the straining-head. With a tension

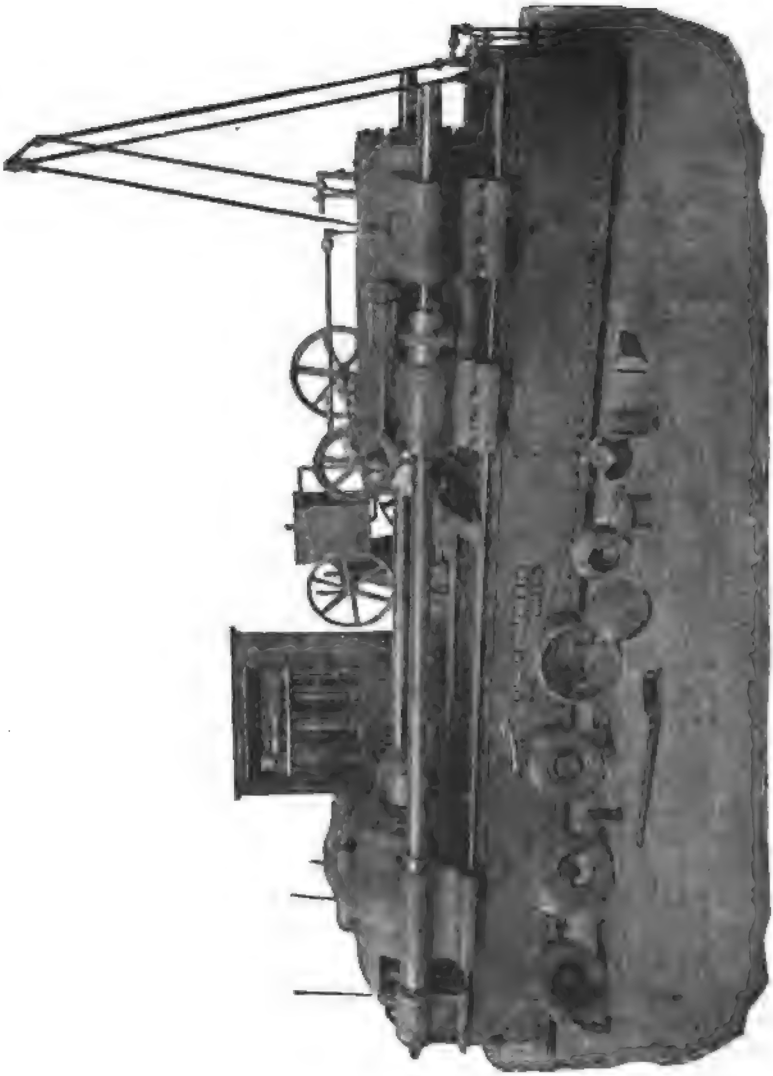


FIG. 268.—Emery Testing-machine of 200,000 lbs. capacity.

specimen between the heads the tendency is to pull them together; the reaction-screws will then be in compression, and the abutment-nuts set up against the rear bearings of the straining-head.

On the sudden breaking of a tension specimen the reaction-screws will suddenly relieve themselves of their compressive stress by pushing the heads of the machine apart with sufficient force to set both in motion along the bed. The motion of the straining-head is gradually destroyed by its own friction against the bed, and in a distance which under all circumstances is well within the clearance-space that the abutment-nuts have between the sleeves for the reaction-screws, while the motion of the weighing-head is also checked by the projecting ends of these screws butting against powerful springs that are locked up under heavy pressure in suitable box-castings rigidly attached to the bed, as clearly seen in the cut, these springs also being strong enough to return the head to its original position. The arrangement of these *recoil-springs* in their boxes and about the projecting ends of the reaction-screws is such that they resist motion of the weighing-head in either direction, and also tend to return it to its original position after a recoil in either direction. Machines of 100,000 lbs. capacity are of simpler construction.

Passing now to the actual construction of the weighing-head, the vital part of the machine, we have in Fig. 267 a horizontal section of a weighing-head showing the arrangement of the hydraulic support and other details. As indicated by the arrows, the machine is supposed to be in operation on a tension specimen, and to facilitate the study of the relation of the various parts the exceedingly small movements that take place between some of them have been enormously exaggerated. The hydraulic support has also been represented in a somewhat simplified form, which would be satisfactory for machines of small capacities but for certain difficulties of handling during construction. The parts marked A and B are the two main castings of the weighing-head, which are securely bolted together, as well by the *reaction-bars* or *screws* C, in a manner clearly shown in the figure, as by several intermediate bolts on their circumferences, as shown in Fig. 268. Thus bolted together they form an exceedingly strong and stiff beam, enabling the stress exerted by the specimen to be returned to the straining-head through the *reaction-bars*. The central member marked D is the *draw-bar*, to which are screwed the two draw-bar heads E and F. The draw-bar with its two heads is kept in position relatively to the main castings A and B by the annular steel plates G, which on their outer edges are centred in and securely clamped to these castings by means of the annular washers H and numerous bolts I, and which in turn centre the draw-bar by their inner circumferences and are securely clamped to its heads by the annular washers J and numerous bolts K. Through their flexibility these steel plates will be seen to allow the draw-bar, with its heads, a certain freedom of motion axially, while they fully maintain its concentric position with the main castings A and B.

The annular castings L and M surrounding the draw-bar between its two heads, and secured to and centred by it by means of annular steel plates P, annular washers, and numerous bolts, in a manner similar to that in which the draw-bar itself has just been described to be secured to the main castings

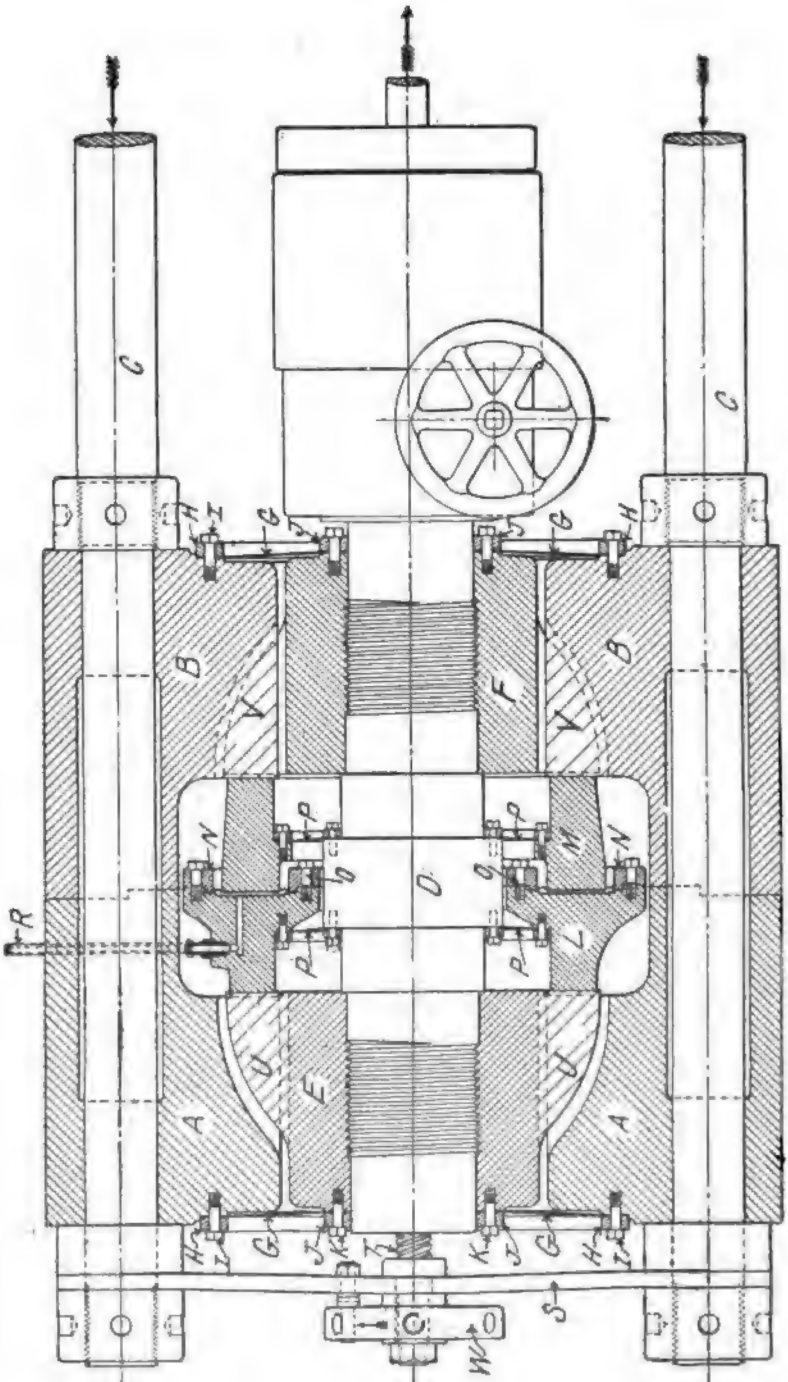


FIG. 267.—Horizontal Section of the Weighing-head of an Emery Testing-machine. (Barth.)

A and B, are respectively the cylinder and plunger (more properly the bearing-plates), or blocks, of the hydraulic support, which is thus seen to be annular in the actual machine, and not circular as the one shown and described in connection with Fig. 265. The parts marked N and O are two annular washers that clamp the edges of the thin brass diaphragms composing the sack to the bearing-block L, the liquid being confined between these diaphragms, as also already described in connection with Fig. 265. The two plates forming this closed sack are not soldered together. They are simply held by the great stress in the bolts which bind the washers N and O so as to maintain a tight joint under the maximum pressure. The pipe forming the communication between the hydraulic support and the reducing-chamber of the scale is marked R.

The draw-bar heads E and F are provided with a number of external projections or spur-ribs, U, extending into cavities in the main castings A and B, as may be seen in the case of the head E and the main casting A. The cavities in A and B leave between them an equal number of projections, V, which extend back into the cavities between the ribs U on the draw-bar heads, as may be seen in the case of the head F and the main casting B. The arrangement of these ribs and cavities, which enables a machine with a single hydraulic support to be used both for tension and compression tests, will be better understood from Fig. 268, in the right-hand view of which this is made perfectly plain.

The manner in which the pull on the draw-bar is transferred by the ribs on the head E to the bearing-block L, from this through the liquid-sack over to the other bearing-block M, and finally from this to the ribs on the main casting B, will now readily be understood, and it will also be seen that a push on the draw-bar, due to a compression test, would in a similar manner be transmitted by the ribs on the head F over to the annular bearing M, from this through the liquid over to the corresponding bearing L, and finally from this on to the ribs on the main casting A.

Owing to supposed internal strains and stresses in the diaphragms of the hydraulic support, it has been found necessary to put about 50 or 60 lbs. per square inch *initial pressure* on the same, the levers of the scale having then first to be balanced by means of a suitable weight that can be slid along a rod attached to the poise-frame lever before beginning a test. This initial pressure on the hydraulic support is obtained in the smaller machines by the device shown in Fig. 267, while in larger machines a very complex device is used to get the requisite pressure, the principle of this, however, being essentially the same. The device as here shown consists of a large flat spring S, attached, in a manner clearly seen in the figure, to the projecting ends of the reaction-bars, and forming the bearing for a screw, T, secured against end motion in this bearing by a collar attached to it on the inside of the spring, and by the capstan-head W fastened to it by a key and a nut in the customary manner on the outside of the spring. This screw fits into a tapped hole in the end of the draw-bar; and it will be understood that if the capstan-

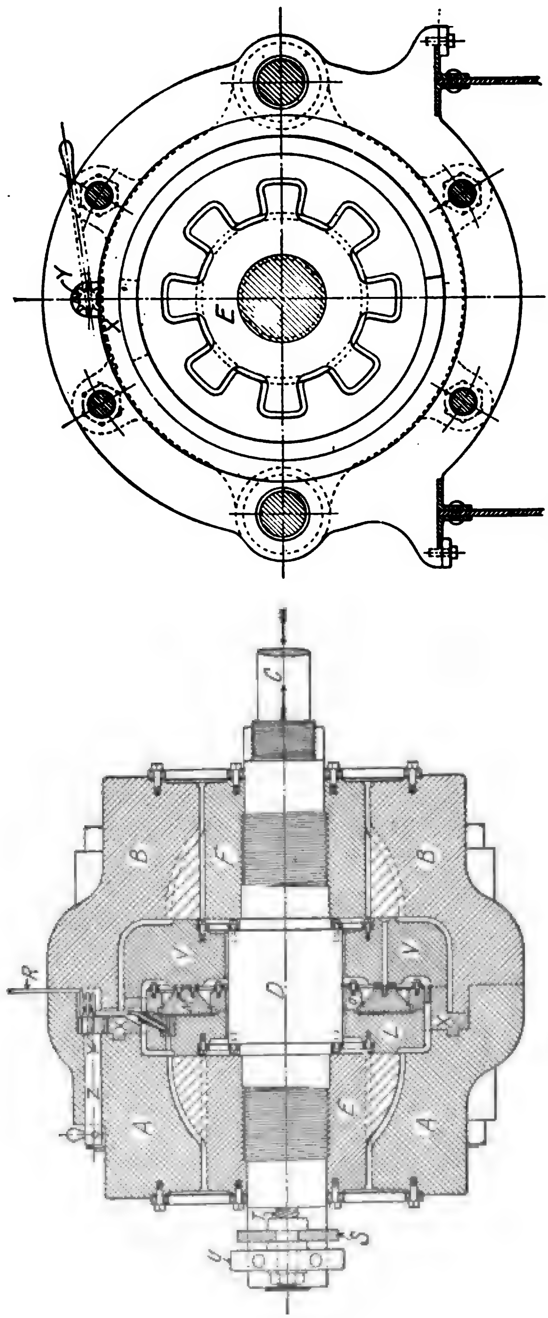


FIG. 268.—Vertical Sections of the Weighing-head of an Emery Testing-machine. (Barth.)

head be turned in the direction of the arrow, the tendency is to move the draw-bar, with it its heads and the whole hydraulic support, away from the spring, until the bearing-block *M* brings up against the ribs on the main casting *B*, further turning producing a bending of the spring in the opposite direction. The resistance of the spring to this bending is then transmitted through the screw *T*, the end of the draw-bar, the ribs on the head *E*, the bearing-block *L*, the liquid, and finally the bearing *M* over to the casting *B*. This condition of affairs is exactly what is brought about before beginning a tension test, for which the machine is supposed to be represented in this figure. A sufficient turning of the capstan-head in the direction opposite to that indicated by the arrow will similarly pull the draw-bar, etc., towards the spring *S* until the cylinder *L* is brought up against the ribs of the main casting *A*; a further turning bending the spring towards the end of the draw-bar. This, with its consequences, is the condition of affairs brought about before beginning a compression test.

It will be seen that the spring *S* is provided with a stop, to match a similar stop on the capstan-head to bring up against, for the purpose of limiting the motion of this latter in either direction, the pitch of the screw *T* being so selected and the whole device so fitted up that the determined amount of bending of the spring in the one or the other direction will produce the desired amount of initial pressure on the diaphragm of the hydraulic support. The total possible movement of the hydraulic support in its chamber, between the attached heads *A* and *B*, is only 0.006 in., so that the maximum movement from a mean position is only 0.003 in.

Fig. 268 is a vertical section of a weighing-head, in which the hydraulic support is shown in its most improved and complete form, the initial pressure device being, however, of the same type as that already described. It will be seen that the annular bearing-block *M* is here not supported on the draw-bar *D* directly, but that it is in the first place fixed in its proper relation to the bearing-block *L*, independently of the draw-bar, by two annular steel plates, rigidly clamped in the usual manner; and that it is also bolted to another large annular casting *V*, which is supported on the draw-bar in the same manner as the bearing-block *L*. In addition to this casting *V* there is also seen another annular casting *X*, a small arc of the circumference of which is provided with gear-teeth meshing with the teeth of the pinion *Y* on the end of the hand-lever shaft *Z*. It may thus to a limited extent be rotated in the one or the other direction, but it is otherwise confined between the main casting *A* and *B*, and also to be centred on *A* by a circular tongue and groove. The plates *X* and *V* are on the figure shown to be in contact, and the contact surfaces are helicoidal or screw-shaped, so that rotation of *X* in one direction tends to force them apart, and to bring *V* up against the ribs of the casting *B*, while rotation of *X* in the opposite direction leaves *V* with a little play between *X* and *B*. The latter of these conditions is brought about by the operator before beginning a compression test, the former before beginning a tension test. The purpose of this arrangement is

to protect the diaphragm of the hydraulic support from the great shock it would otherwise receive by the sudden release of the stresses on the various parts of the weighing-head, on the sudden breaking of large specimens when a powerful recoil of the draw-bar occurs. With the annular *anvil V* set up tight against *B* by the annular *wedge X*, the energy of this recoil is trans-

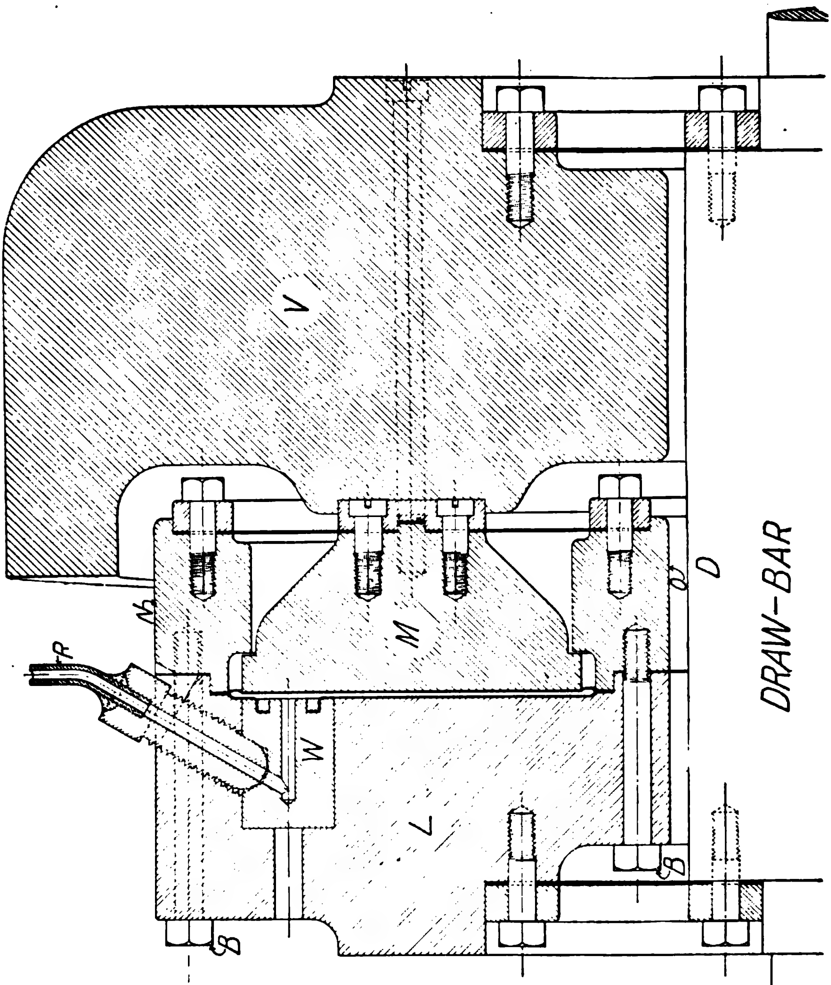


FIG. 269.—Enlarged View of a Longitudinal Section of the Hydraulic Support on one (side of the Draw-bar in an Emery Testing machine. (Barth.)

mitted directly over to the main casting *A* without passing through the liquid support and the diaphragm, on which an exceedingly high pressure would otherwise be produced, which if frequently repeated would finally destroy it.

In Fig. 269 is shown on a larger scale a section of the annular hydraulic support on one side of the draw-bar only, which, without any further explana-

tion, will serve to give a clearer idea of the detailed arrangement of the diaphragms and the surrounding parts, and also the manner in which the pipe R, which forms the communication between the hydraulic support and

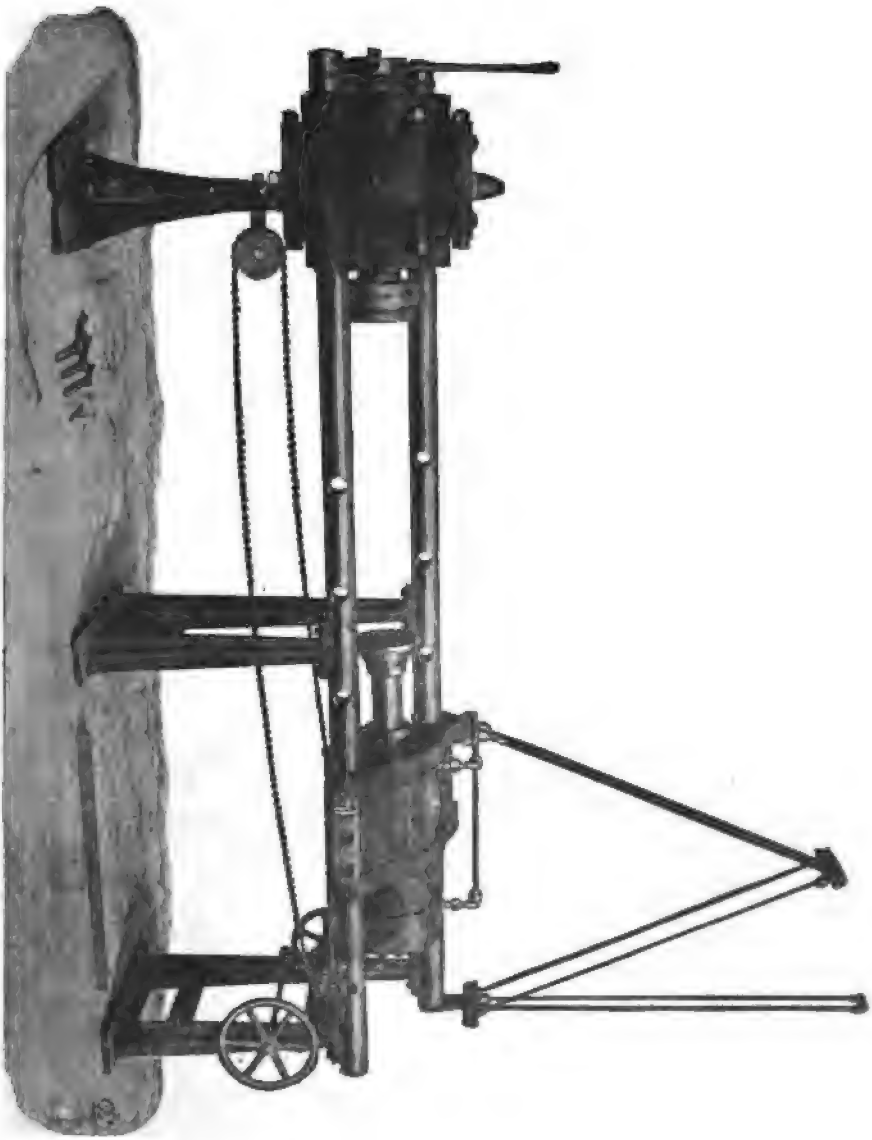


FIG. 270.—Emery Testing-machine of 100,000 lbs. capacity.

the corresponding reducing-chamber on the scale, is attached to the former. The two plates which form the sack lie flat against the bearing-blocks, underlaid, however, by connected grooves, their edges being held so tight by

the bolts B as to prevent all leakage. The plate on the left side of this sack is cut away and spun into an annular pocket in the auxiliary block W, and it is made tight to it by running in a solder. To this auxiliary block is now attached the connecting pipe R as shown. This pipe is first soldered to a screw-plug which has a spherical front which bears on the conical bottom

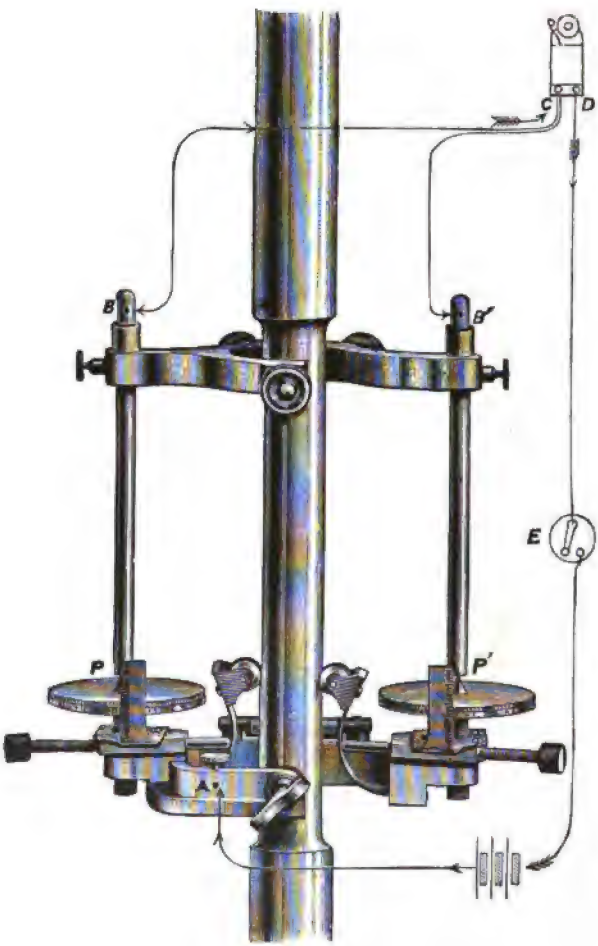


FIG. 271.—The Riehlé-Marshall Extensometer.

of the opening in W, so that by screwing up hard a perfect joint is made by the elastic deformation of the metal at the surface of contact.

In Fig. 270 is shown the form given to this machine when made with a capacity of 100,000 lbs.

274. Extensometers.—There are three general types of extensometers in common use, viz., Double Micrometer-screws with Electric Contact;

Friction-rollers with Dial Indicators; and Bauschinger's Mirror Apparatus.

The micrometer-screw extensometer was developed and perfected by Mr. C. A. Marshall, M. Am. Soc. C. E.,* and it is now manufactured, with some improvements, by Riehlé Bros. as shown in Fig. 271. The essential features of all these extensometers are:

1. Measurements are taken on opposite sides of the specimen, between symmetrically placed points on rigid collars which are attached to the specimen by screw-points or knife-edges lying in two transverse planes a known distance apart.

2. The measurements must be taken to the nearest $\frac{1}{10000}$ inch.

3. The apparatus must be removable without releasing the load on the specimen.

In *the Marshall Instrument* the collars are open, thus enabling them to be removed, and also giving to them a sufficient spring to take up the reduction in the diameter of the specimen as it elongates. The elongations are read on micrometer-screws to 0.0001 inch, the contact being determined by the ringing of an electric bell on the closing of a circuit by the contact. With a low but constant current this contact-distance is found to be constant within the limit of reading given above.† Both screws are read after each increment of loading, and the average movement taken as the stretch of the specimen. This is a most excellent and delicate instrument and is very largely used. It has the advantage of a more positive and direct measurement of the deformation of the specimen than either of the other forms.

The extensometer having *friction-rollers with dial-indicators*, shown in Fig. 272, is a modification of one of Bauschinger's forms, as made and used by the author.‡ It operates by means of two axles, having friction-rollers at one end and a vernier-needle at the other which moves over a graduated dial. The friction-roller is just 0.5 inch in circumference, and the dial is graduated to 500 divisions. The vernier on the end of the needle reads readily to 0.1 of

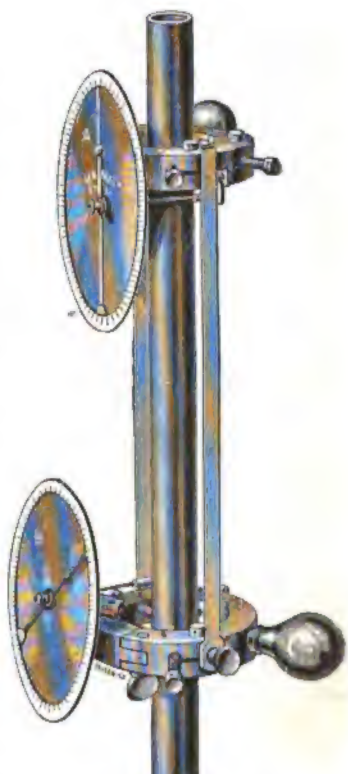


FIG. 272.—The Author's Extensometer.

* A brilliant young engineer, a friend and college-mate of the author's, who lost his life in the great Johnstown flood, 1889.

† By using a resistance relay in the circuit a strong current may be made to pass through the bell, and a weak one through the contact points.

‡ Manufactured by Mahn & Co. of St. Louis, Mo.

a division, thus giving readings to 0.0001 inch. The collars are attached by three screws, and removed by opening them by a hinge movement. One of the screws has a spring bearing in the collar to take up the shrinkage of the specimen when under stress.

The friction-rollers are actuated by means of two arms which have a spring bearing upon them, the opposite ends being rigidly attached to the opposite collar in each case. Thus while the needles move in the same direction any bending of the specimen, or angular movement of the collars with respect to each other, is eliminated in the mean of the two readings, the same as with the Marshall apparatus. The needles are delicately mounted so as to have very little friction, and experience shows that the friction-contact is entirely reliable.

The advantages of this form of extensometer are:

1. It shows by its movements the deforming action of the specimen, which is a great advantage for students.

2. It is equally suited to measure large deformations beyond the elastic limit as it is to measure the extremely small movements inside that limit.

3. It is equally adapted to compressive and tensile tests.

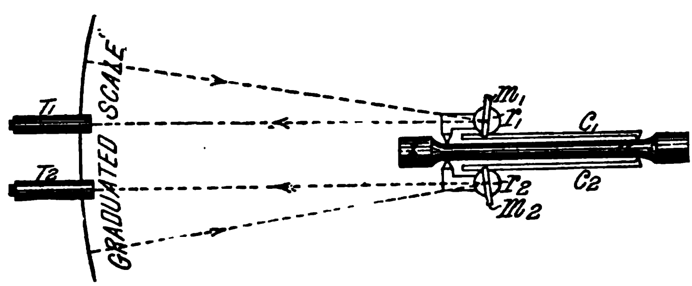


FIG. 278.—Diagrammatic Plan of Bauschinger's Mirror Extensometer.

4. It is adapted to all lengths of specimens by simply changing the side arms, several pairs of which go with the instrument.*

5. On releasing the load it shows the permanent set without any manipulation.

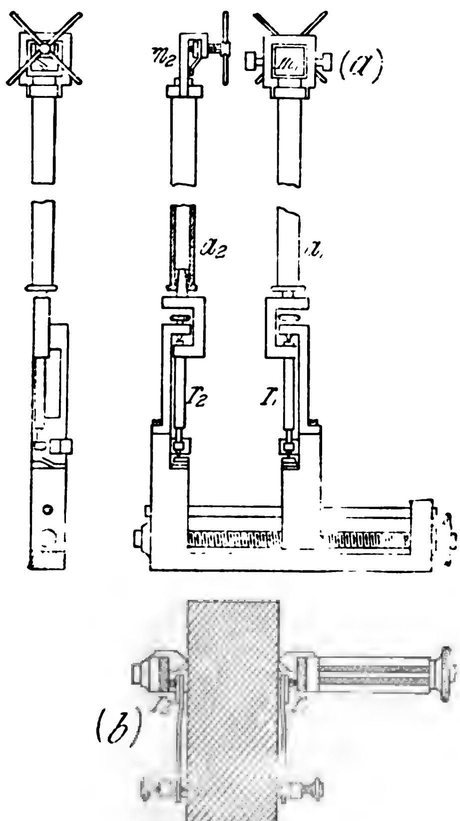
6. It never needs to be touched by the observer during a test, which is a great advantage in making such delicate measurements.

7. After passing the elastic limit, and under a given constant load, the continued movement of the needle indicates the time-effect of such loads and when such cold-flowing has practically ceased.

If the specimen should unexpectedly break with the apparatus on, no great harm results. At most some of the clamping-screws may be bent, but these cost little to renew.

* The author has used it successfully for observing the effects of moving loads on bridge members, with arms five feet long, covered with thin rubber at their roller ends, and specially made U-shaped clamps instead of collars.

Bauschinger's Mirror Apparatus is shown in Figs. 273 and 274. The specimen is clamped at two points, as shown at (*b*), Fig. 274, and at *a* and *b*, Fig. 273. The stretch of the specimen is fully represented by the turning of the friction-rollers r_1 and r_2, these being rigidly attached to the mirrors m_1 and m_2 through the arms a_1 and a_2. The screws set back of the mirrors



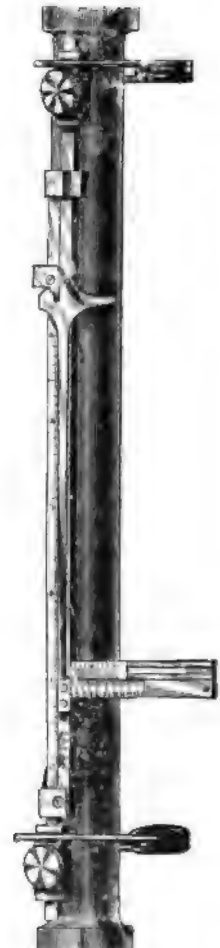


FIG. 274.—Bauschinger's Mirror Extensometer Apparatus. FIG. 275.

are used to adjust them to a zero-reading on the scale, which is reflected into telescopes as shown in Fig. 273.

For simply observing stretch for the purpose of detecting the elastic limit with reasonable accuracy, the Paine extensometer, Fig. 275,* may be used

* Designed by W. H. Paine, M. Am. Soc. C. E., and used by him for finding the elastic limit of the steel wire used on the New York-Brooklyn suspension bridge. Made now by Riehlé Bros.

with advantage. Its multiplication is usually made about 20 to 1, and it may be read by vernier to 0.0001 inch, though it is usually made to read only to 0.001 inch. This instrument has also been used to obtain the stretch of bridge members under moving loads. As it measures stretch on only one side of the specimen, its indications must not be accepted as absolute, especially inside the elastic limit, while its capacity is very small beyond the elastic limit.

A very simple and inexpensive apparatus which may be made to give excellent results is shown in Fig. 276. If care be taken to secure a practi-

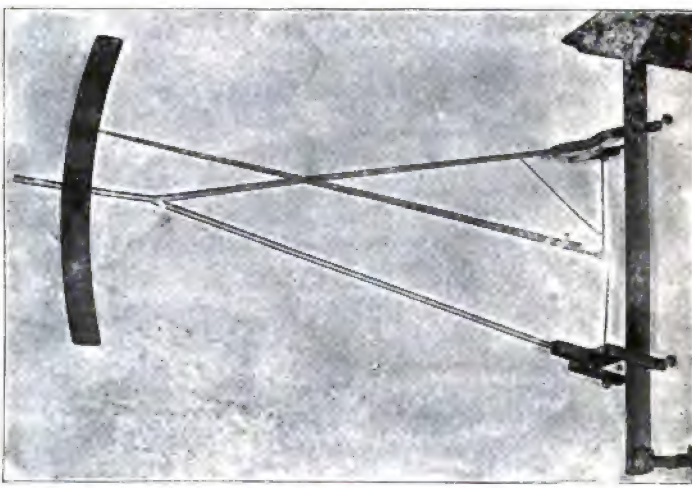


FIG. 276.—Extensometer used in Yorkshire College, Leeds. (From *Engineering*, Sept. 11, 1896.)

cally constant length of the short arm of the indicator, and if the legs of the main frame which carry the graduated arc are equally flexible, fairly accurate readings can be obtained.

275. Autographic Stress-diagram Appliances.—These fall into two general classes:

1. Those in which the load coordinate is recorded through a movement of the poise on the weighing-beam.

2. Those in which the load coordinate is recorded through the lifting of the weighing-beam against the increasing resistance of a calibrated spring attached to its free end.

The deformation coordinate is in all cases multiplied either by levers or by the principle of the cone-pulley. The paper is usually attached to a cylinder, although it has sometimes been attached to a plane board. The pencil usually moves in a straight line, indicating one of the two coordinates, while the cylinder (or board) moves to register the other function, and it matters not which of the two movements is made by the deformation of the

specimen and which by the increasing load. The location of the paper and its mounting is a matter of convenience simply. The cords (or wires) which are to transmit the stretch of the specimen must form a pair, symmetrically placed on opposite sides of the specimen; they must be attached to one collar and pass through pulleys similarly placed on the other. They should then pass off in a plane at right angles to the specimen * and connect with the ends of an "evener" (lever), to the centre of which is attached the single cord which passes either to the pencil-holder or to the cylinder which carries the paper. If cords are used, they should be such as do not stretch appreciably for such changes of stress as occur in them during the test.

One method of mounting these parts is shown in Fig. 256 (Olsen's), and another form in Fig. 258 (Riehlé's). In both cases the pencil is moved by the poise by a reducing-gear, and the cylinder is moved by the deformation of the specimen by a multiplying-gear.

Figs. 277 and 278 show two improvised forms of autographic diagram

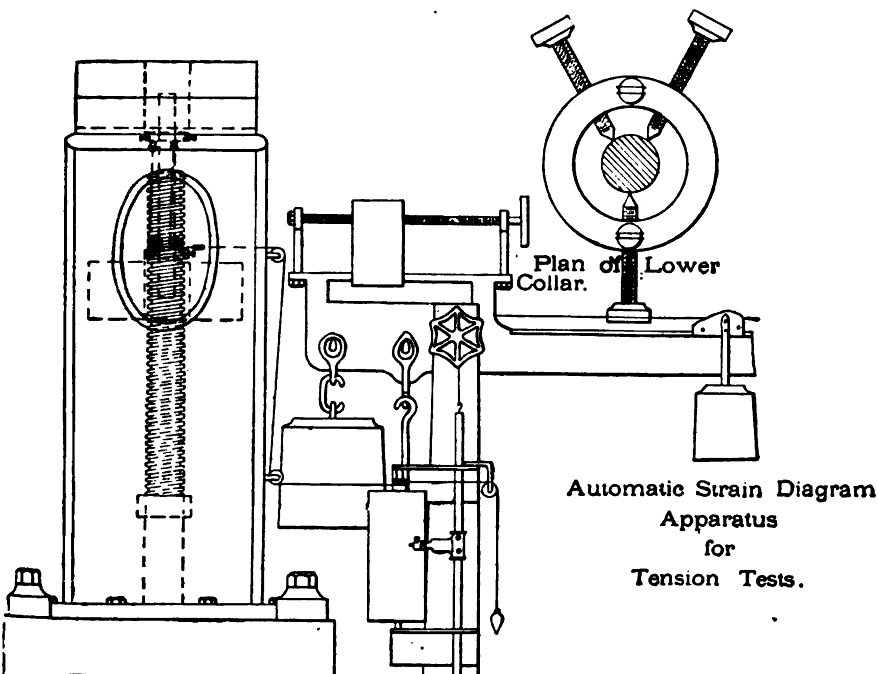


FIG. 277.

apparatus which the author devised and had attached to his 100,000-lb. hydraulic testing-machine, for tension and punching tests respectively.

* This is necessary in order that the stretch of the specimen may be fully represented in the shortening up of the cord. The cords should therefore be attached to the moving end of the specimen.

Here the pencil is moved by means of a fine wire which coils over the small spindle to which the poise driving-pulley is attached, while the cylinder is

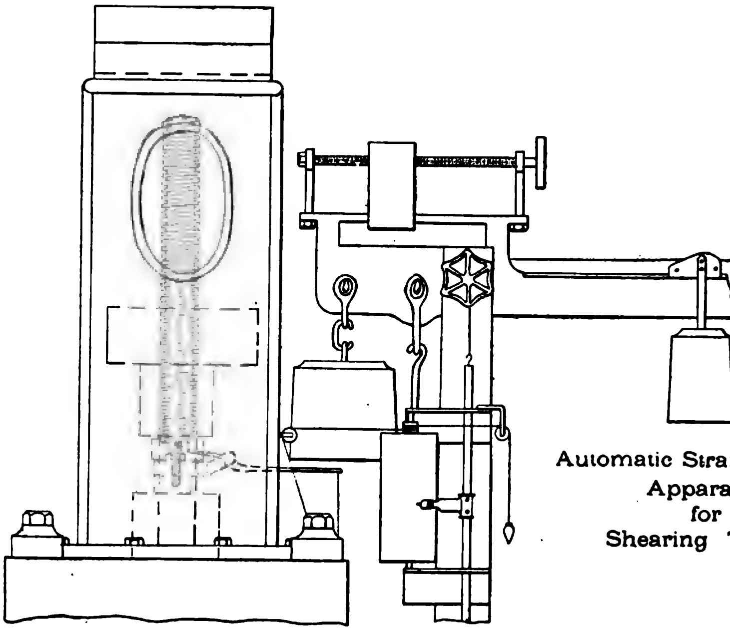


FIG. 278.

turned by the cord from the specimen runs upon a small drum at top.* This cord is kept taut by means of a small plumb-bob. The clamp-screws in the collars are set so as to take hold of a square or rectangular specimen as well as a round one. The second form was made for a series of punching tests of steel plates.

A great defect of all the autographic appliances given above is that they do not readily give the last end of the curve after passing the maximum load, although they would do this if the poise should be run back so as to keep the beam in balance at all times. This is a difficult feat with both the hand- and the electrically-controlled movement of the poise, and the result is that this part of the diagram is usually worthless.

In *the Gray Extensometer Apparatus*,† however, Fig. 279, this part of the curve is obtained perfectly, for the weighing-beam is at all times in perfect balance, since it pulls upon a calibrated spring. Here the pencils are moved at two rates of speed by the deformation of the specimen, and the cylinder is turned by the lifting or dropping of the weighing-beam. The

* This cord should lead off from the specimen at the upper (fixed) cross-head instead of from the lower one. The drawing is erroneous in this particular.

† Designed by Prof. Thos. Gray, and manufactured by Riehlé Bros.

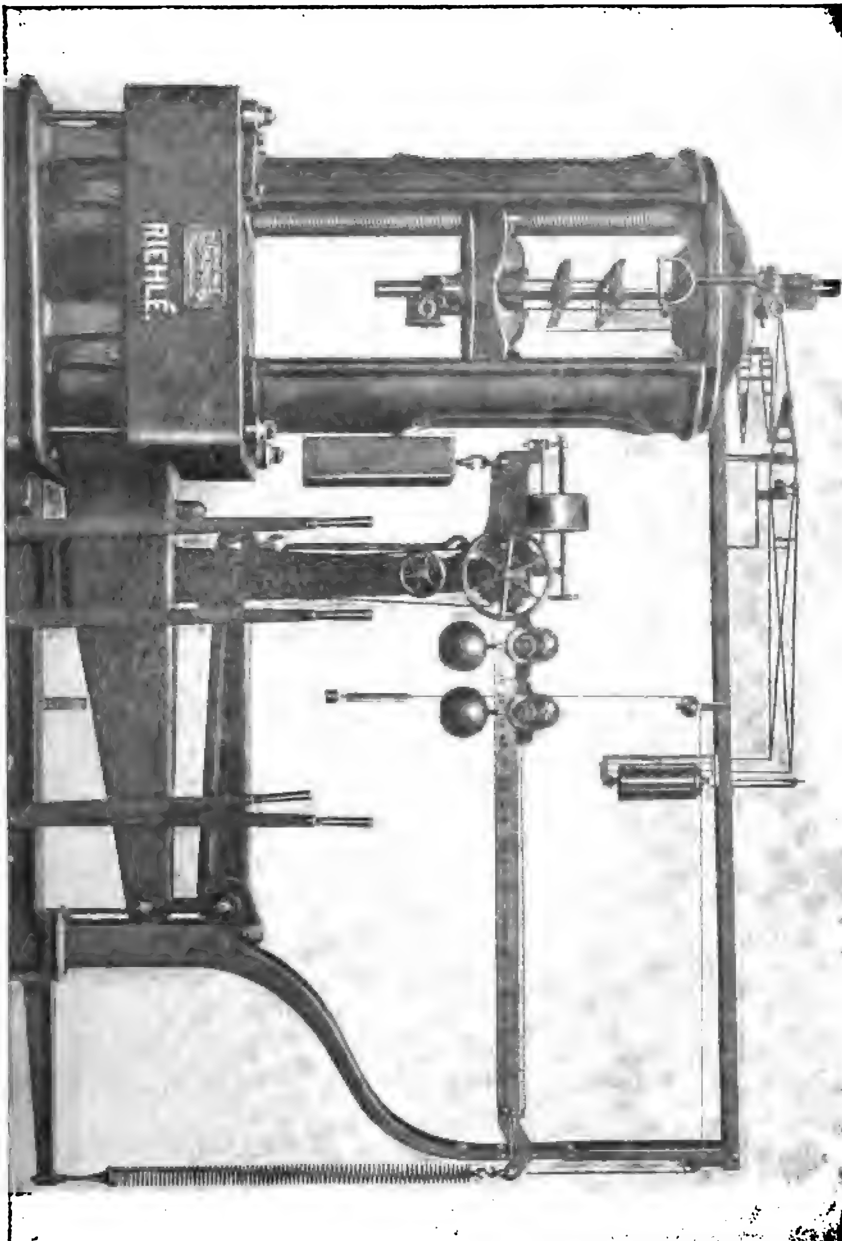


FIG. 229.—Showing the Gray Extensometer Apparatus mounted on a 200,000 lb Testing machine.

ong trussed lever at top gives a movement to one of the pencils of from one to two times the actual deformation between the collars, while the other trussed lever may give to the other pencil a movement from one hundred to five hundred times the actual relative movement of the collars, depending in each case on the link connections which are made preparatory to starting the test. Furthermore, both pencils operate inside the elastic limit and some distance beyond, when the one moving more rapidly is automatically thrown out of gear, while the other pencil proceeds to record the complete diagram on the smaller scale. The result is a double stress-diagram, such as shown in Fig. 5. The diagrams shown in Fig. 280 have been photographically reduced directly from autographic diagrams made by this appli-

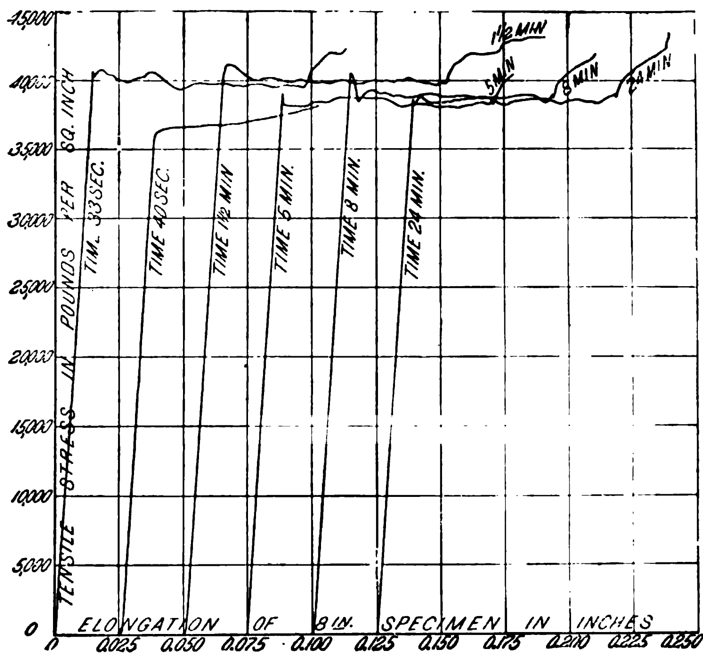


FIG. 280.—Tension-stress Diagrams of Low Carbon-steel automatically recorded by the Gray Apparatus. (Made by Prof. Gray for the author.)

ance for the author by Prof. Gray himself. They do not extend beyond the yield-point stage of the test.

Mr. Olsen's new Micrometer Autographic Attachment forms a supplement to his general stress-diagram apparatus, for the purpose of making a diagram inside of and somewhat beyond the elastic limit, in which the stretch of the specimen is magnified five hundred times. He accomplishes this by revolving the drum one hundred times as fast inside the elastic limit as is done beyond that limit, the stretch of the specimen here being greatly multiplied by a micrometer-screw and its accompanying gearing shown in Fig. 281. When-

ever the collars separate, the lower pair of fingers (being weighted) drop with the lower collar and so break an electric spring-contact shown on the left of Fig. 281, which sets in motion both the drum carrying the paper (not shown here) and the micrometer-screw and its gearing, which at once closes the circuit by raising the outer end of the lever to which the spring-contact

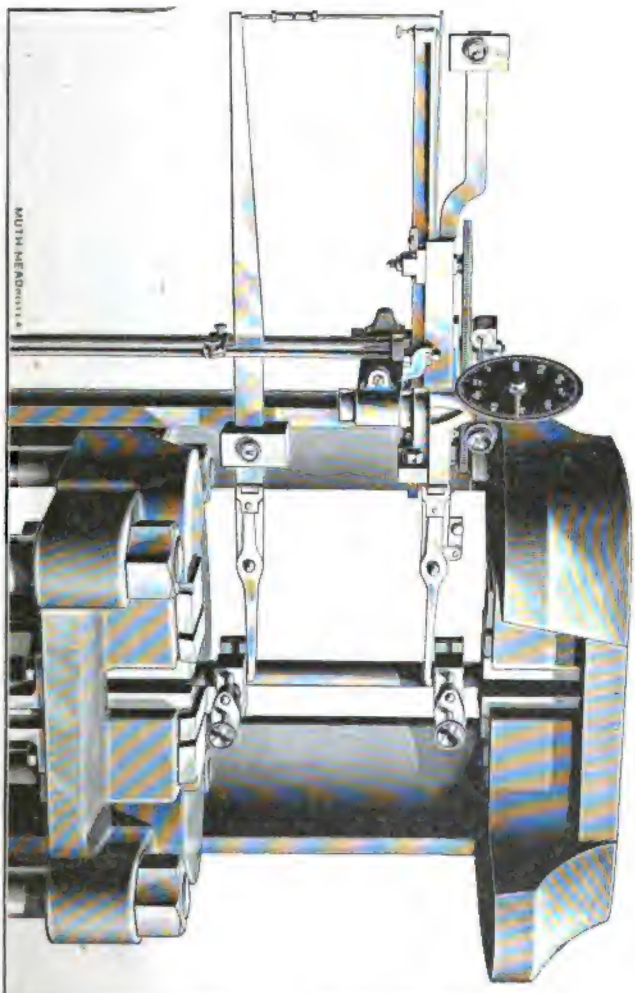


Fig. 281.

is attached. A very large circumferential motion can thus be obtained for a very small movement of collars (500 to 1), and this can be conveyed by a positive connection to the drum of the autographic apparatus. When the elastic limit has been passed this part of the apparatus is thrown out of gear, the drum set back to its proper position under the pencil for the small-scale

diagram (deformation 5 to 1), and the test proceeds to its completion at the final rupture of the specimen. The poise is moved either forward or backward at pleasure by making the proper electric connections, or both these connections can be made at once, in which case the poise moves forward when the beam is up, and backwards when it is down. The last part of the diagram

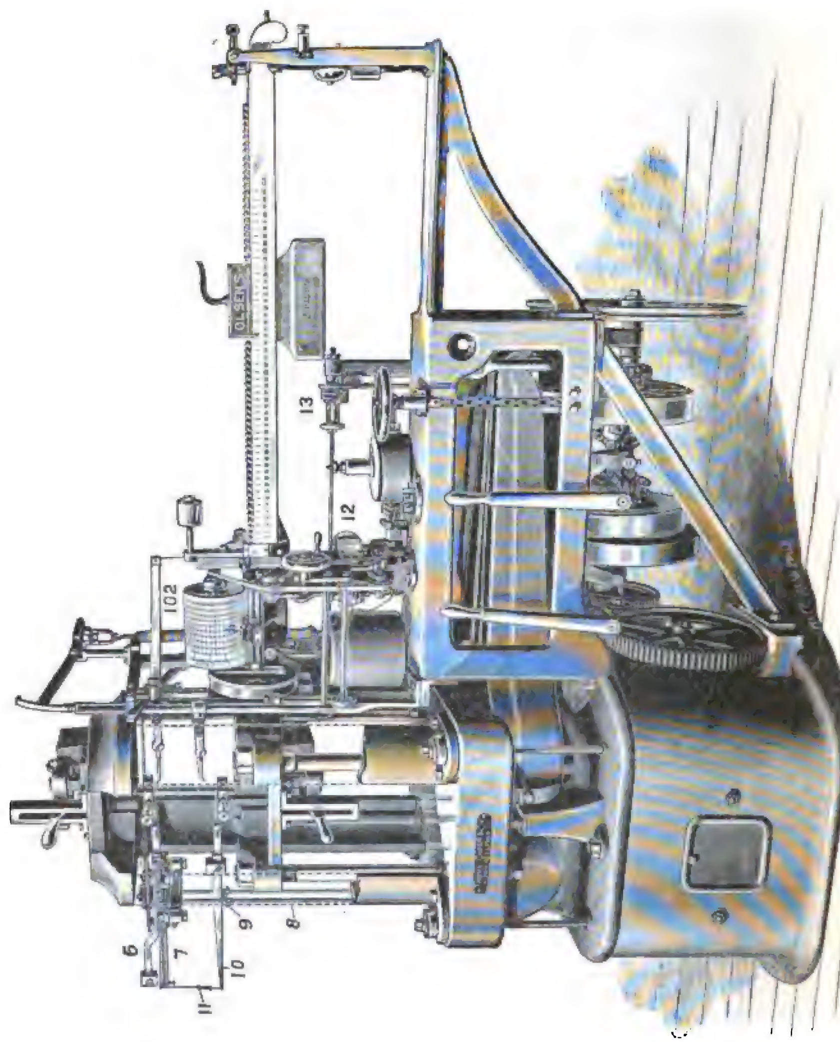


FIG. 282.

can thus be obtained. The entire machine, with both the large- and the small-scale diagram apparatus, is shown in Fig. 282.

276. Micrometer-callipers.—In Figs. 283, 284, and 285 are shown three forms of micrometer-callipers, one or more of which are necessary for accu-

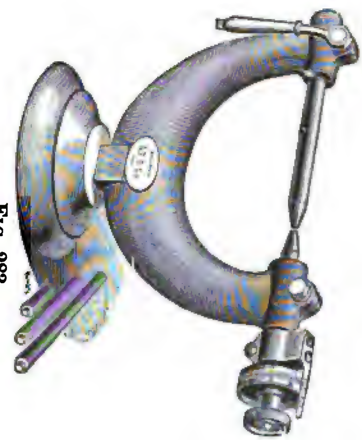


FIG. 283.

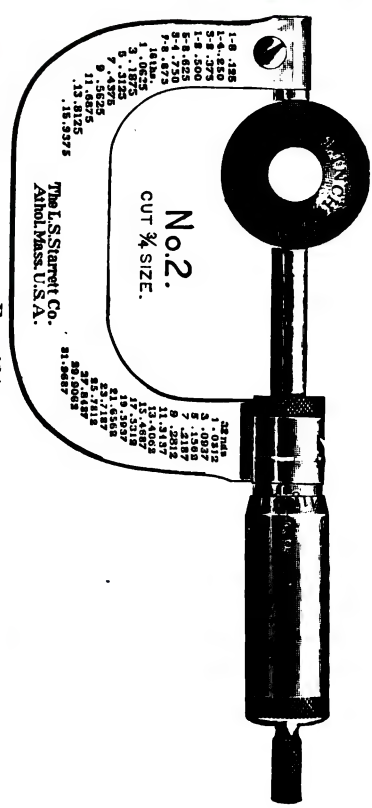


FIG. 284.

FIG. 285.

rately measuring the dimensions of test specimens. The form shown in Fig. 283 * spans 8 inches; that shown in Fig. 284 spans 2 inches, and that in Fig. 285 spans $2\frac{1}{2}$ inches, all of them reading to 0.0001 inch. The advantage of the last form is that one may be set for the width and the other for the thickness of plates, thus saving much running back and forth of the micrometer-screw.

277. Gauging-implements.—It is common to lay off the test specimen into 1-inch divisions either by scratch-awl or centre-punch marks. For the

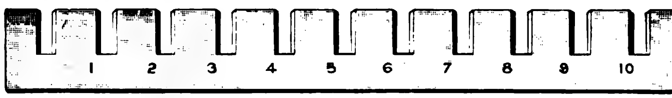


FIG. 286.

former (used on plate specimens) the laying-off gauge shown in Fig. 286 is used, and for the latter (used on round specimens) the double-pointed

FIG. 287.

centre-punch shown in Fig. 287 is most convenient. Some such instruments are essential to accurate work, and they also are great time-savers.

---

* Designed by Prof. Sweet and made by the Syracuse Twist Drill Co., Syracuse, N. Y.

CHAPTER XVI.

COMPRESSION TESTS.

278. Objects of Compression Tests.—While tension tests are made for the purpose of determining many of the more significant mechanical properties of the malleable metals, compression tests are made to determine resistance to compression alone. In Chapter III it was shown that the materials of construction divide themselves into two general classes with respect to their manner of failure in compressive tests, these two classes being called plastic or viscous materials, such as the malleable metals, and brittle or comminuible materials, such as cast iron, stone, brick, etc.

When testing plastic materials in compression the "apparent elastic limit" must be regarded as the ultimate strength; and since this limit in compression is in nearly all cases the same as it is in tension, it is commonly taken as the same, and no compression tests are made on such materials except when made up into full-sized columns. The compression tests of these are known as "tests of columns," rather than "compression tests" of that material.

Brittle materials are tested in compression to determine their resistance to crushing.

279. Compression-test Specimens.—In the case of metals the test specimens can be turned or shaped accurately, but in the case of stone, cement, concrete, brick, etc., it is not practicable to obtain perfectly true specimens, and hence some suitable provision must be made for these when placing them in the testing-machine. For such materials the form of specimen hitherto almost universally employed has been that of the cube. In Chapter III it was shown that this form is too short to give a normal failure; that the length in the direction of the applied load should be at least 1½ times the least lateral dimension. It is probable, however, that compression tests will continue to be made on cubical forms, for the reason, that the results may thus be comparable with those hitherto obtained and published. The general relation between the strength of cubes and of prisms of various ratios of height to least breadth, for sandstone, is shown in Fig. 17, Chapter III.

While perfectly true and parallel surfaces cannot usually be obtained, they should be made as nearly so as possible. This can be done at a small cost if stone-grinding works are at hand, or if such a special grinding-machine is available as that shown in Fig. 288.

The test specimen should be very nearly prismatic, since when the sides protrude much beyond the bearing-surfaces the specimen is strengthened as shown in Fig. 18.

FIG. 288. — An Abrasion Testing-machine.

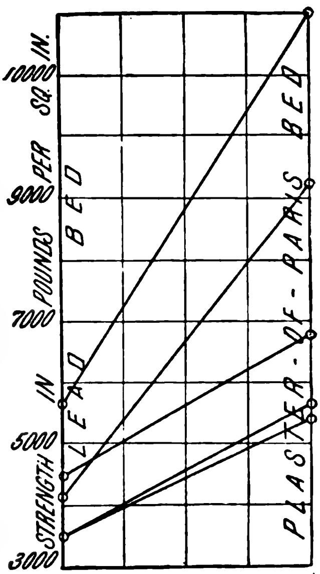


FIG. 289.—Showing the Effect of Bedding on the Strength of Sandstone. (*Inst. Civ. Engrs.*, vol. CVII.)

280. Bedding the Specimen in the Testing-machine.—If the specimen has not true and parallel beds, it is necessary to embed the specimen in plaster of paris. This is done by inserting sized paper between the plaster of paris and the specimen to prevent the absorption of water by the specimen, which invariably weakens it if it has a high absorbing capacity. A small load is brought upon the specimen while the plaster beds are soft, and this is left on for some ten minutes or longer, till the plaster has hardened, when the test proceeds to failure. *Great care must always be taken to put the test specimen accurately in the axis of the testing-machine.* Compression tests probably more often give erroneous results from not having done this than from any other cause.

If the specimen has true and parallel beds, then it may be placed directly between steel plates, or between the machine cross-heads, if these are true and smooth. Or single thicknesses of tar-board may be employed. In any case no bedding material must be used which will flow, like lead, or spread, like wood, when the load comes on. This causes the specimen to split up and to fail in detail. (See Fig. 289.)

An infallible test of proper bedding and placing in the testing-machine is the manner of failure of the specimen. If it spalls off on the sides (especially if it spalls mostly on one side) before final crushing down, some-

thing is wrong. It should spall very little, and should crush down suddenly, with a great explosive sound, and fly over the room.

An adjustable bearing-plate at one or at both ends of the specimen is desirable, but not strictly necessary if care be taken to secure in other ways a true initial bearing.

281. Compression-test Machines.—The universal machines shown in Figs. 256, 257, 258, 259, 260, 266, and 270 are all adapted to the making of compression tests as well as tests in tension. In Fig. 377 is shown a machine for compression (cement) tests only, and the author has had constructed a machine for testing timber columns, with a capacity of 1,000,000 lbs., which works in compression only, but in general all the compression machines used in America are of the universal type.

282. Compressometers.—Since compression-test specimens are generally very short, the ordinary appliances used in tension tests for measuring

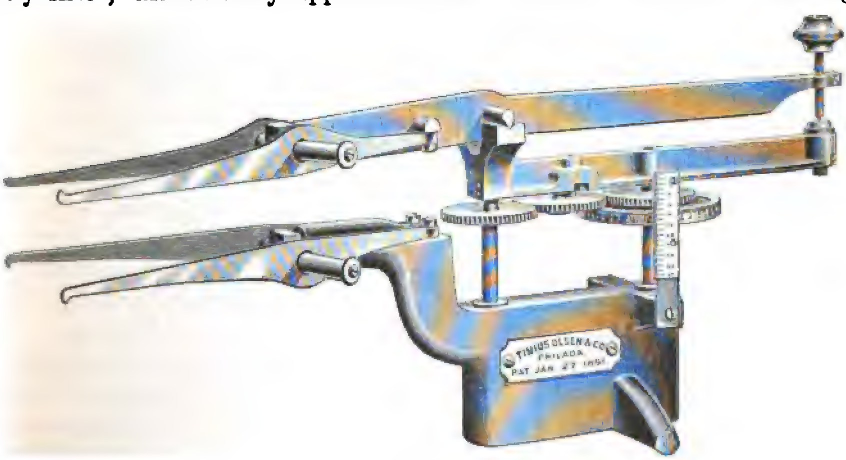


FIG. 290.

deformations cannot be employed. In Fig. 290 a very convenient compressometer is shown, which is adjustable to varying heights of specimen by moving the geared pair of screws, and to specimens of excessive height by introducing new sets of screw-stems. The bearing-points are in pairs, mounted on rockers, so that any unsymmetrical movement is provided for and eliminated. It reads to $\frac{1}{10000}$ inch by electric contact at the right end, under the set-screw there. The deformation of the specimen breaks this contact, and by turning the micrometer-screw the contact is made, this being indicated by the ringing of a magneto-bell.

For large specimens a form like that of Prof. C. Bach, Fig. 291, may be used.* This measuring device consists of two rings, *AA* (on top) and *BB* (below), each of which is fastened to the specimen by means of four screws,

* The description here given is taken from *Zeits. d. Ver. Deutscher Ingenieure,* April 27, 1895.

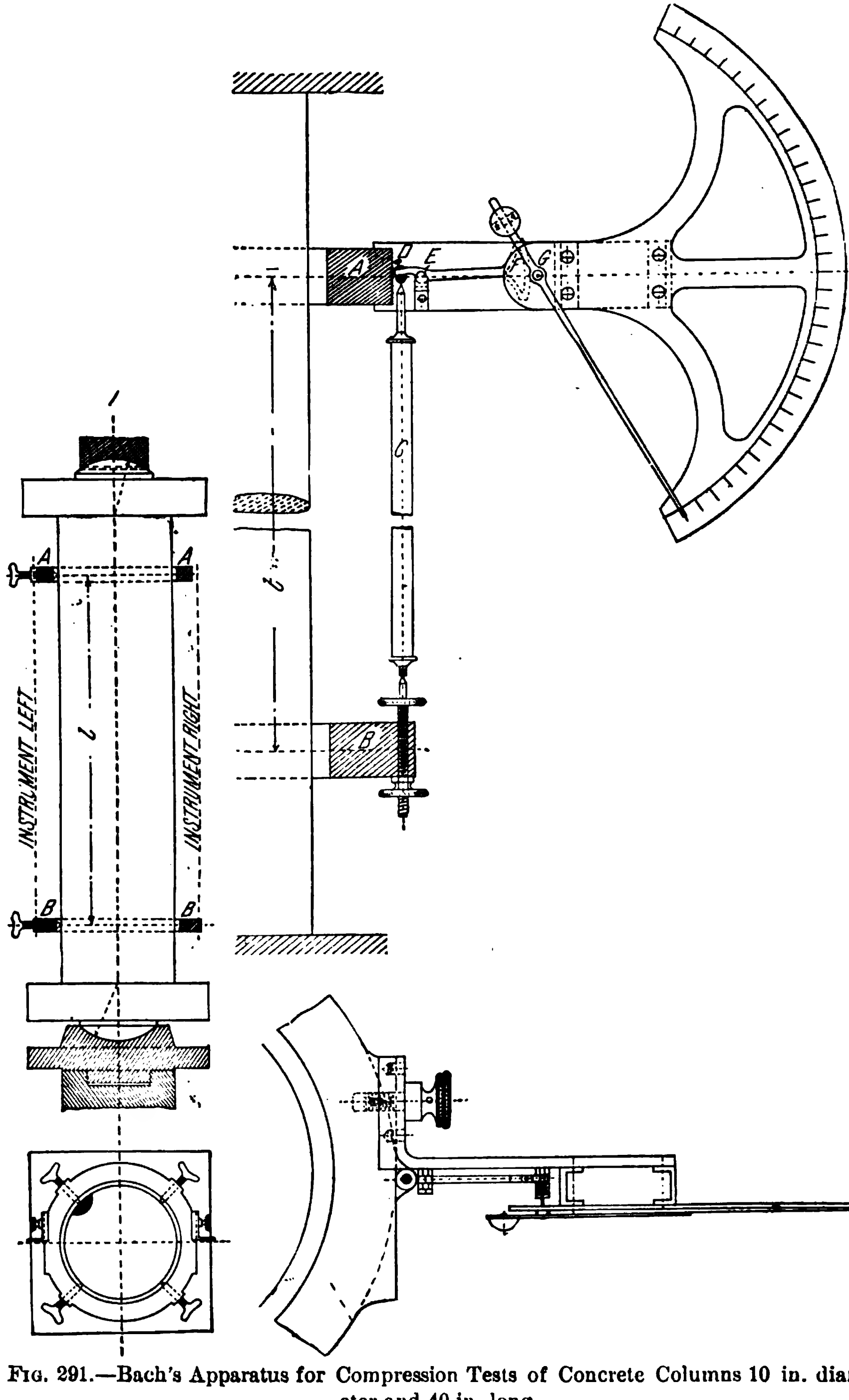


FIG. 291.—Bach's Apparatus for Compression Tests of Concrete Columns 10 in. diameter and 40 in. long.

being at right angles to each other at any convenient distance apart. (The apparatus shown in the figure was used on concrete cylinders 10 inches in diameter and 40 inches long, the rings being 30 inches apart.)

The measuring apparatus is shown on the right. If a compression of the test specimen occurs, the upper terminal point of the rod, C, which length remains the same, will move upwards a distance equal to the distortion of the specimen, thus causing the lever DEF to turn around its axis at E, and to carry along with it, by the small metallic thin ribbon fastened on its segmental end F, the axis G, on which the indicator is fastened, which latter runs along the arc graduation. The indicator is not pointed at the end, but flat, and upon it is an index-line, as may be seen in the drawing. The ratio is made so that 1 mm. compression of the test specimen equals 300 mm. distance on the arc scale. Since this can be read to $\frac{1}{10}$ mm., the distortion of the measured distance can be read to $\frac{1}{3000}$ mm. The only new feature of this instrument is the use of the thin metallic ribbon in place of gearing.

The disadvantage of employing a rack and pinion was that the loads had to be varied by loading and unloading; the least lost motion would produce serious errors. Furthermore, the transmission proportion is dependent on the form of the tooth, for, being obliged to make the teeth so very small, we cannot depend on forming them sharp enough to maintain a constant transmission proportion.*

There should always be two such measuring instruments, set opposite each other, as shown in the general view on the left. By this method the measurement of the deformation takes place at two diametrically opposite points, the mean of the two readings being used.†

For measuring given percentages of compression deformation of wood blocks of varying thicknesses, for instance, the author devised the apparatus shown in Fig. 292. Here a metal point, attached to a sleeve, moves on an adjustable inclined arm, so bent that the point moves on a line through the hinge in the plane of the flat base of the apparatus. The making of the contact at the point rings an electric bell, and the free movement of this point is interrupted by spring stops at such percentages of distortion as are to be observed (with the U. S. timber tests, in compression across the grain, these observed deformations were 3 per cent and 15 per cent of the thickness of the block). For a specimen of any height it is only necessary to move the point to its outer limit, raise it into contact with the upper compression-head of the machine, and tighten the thumb-nut. Then slide the point back to the first stop and proceed to load the specimen. When the bell rings note the load for that limit, and slip the point to the next stop, etc.

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* Where simple friction of a bar on a rolling pinion is relied on to move the indicator-needle, great care must be exercised, especially when loads are applied or released suddenly.

† These instruments are made by C. Klebe of Munich, Germany.

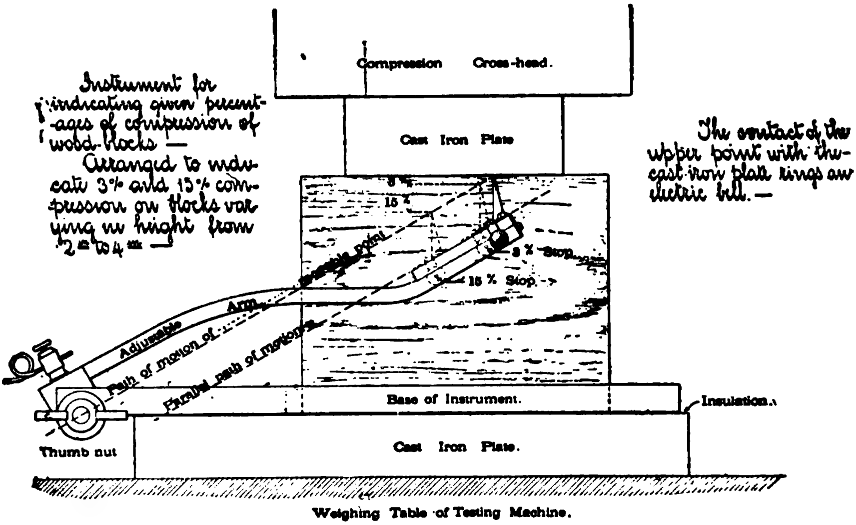

FIG. 292.—Compressometer designed by the Author to Indicate two Conventional Limits of Deformation (3% and 15%) of Wood Blocks when Tested Across the Grain.

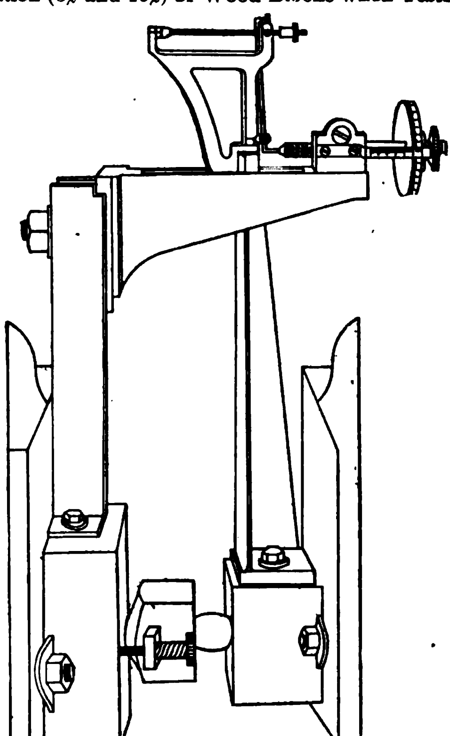

FIG. 293.—Tetmajer's Compressometer for very short Specimens Tested in a Horizontal Machine. (*Zurich Laboratory Communications*, vol. IV.)

283. Tetmajer's Apparatus for Short Specimens.—In Fig. 293 is shown Prof. Tetmajer's apparatus for measuring the deformation of very short specimens.* It is used in a horizontal (Werder) machine, and stands upright as in the figure. The compression of the specimen is taken up on a micrometer-screw which operates on the short arm of the indicator, the long arm of which actuates the upper, balanced, horizontal lever, thus bringing it to its zero position as shown by graduation lines on its left-hand end, and on the adjoining fixed portion of the frame. Because of the measurements being taken on one side only, and that a long distance from the specimen, the readings, while giving true relative motion, would probably not be true absolutely. Tetmajer used it mainly to determine elastic limits. His readings were taken to 0.0001 inch, and great care was taken in centring the specimen in the testing-machine.

284. Compression Tests of Columns.†—Since the strength of a long column consists in its resistance to bending, rather than in its resistance to crushing, it follows that the strength of a straight column is a function of—

1. The elastic rigidity (modulus of elasticity) of the material, E.
2. The ratio of its length, l, to the rigidity function of its cross-section, which is the radius of gyration, r, that is, $\frac{l}{r}$.
3. The character of its end bearings as to their tendenoy to hold the column to its original position, and
4. The eccentricity of the loading.

For a straight column, symmetrically loaded, supported at its gravity axis, and so as to be *perfectly free* to bend, and for a ratio of $\frac{l}{r}$ sufficiently large, it is shown in works on mechanics (and in the author's "Modern Framed Structures") that *Euler's Formula* gives the strength of the column. This formula is

$$p = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2},\ddagger$$

where p = ultimate strength of the column, in pounds per square inch;
E = modulus of elasticity, in pounds per square inch;
l = length of the column between the pivot-bearings, in inches;

* Described in Tetmajer's *Communications*, vol IV. (1890).

† It does not fall within the province of this work to enter into a general discussion of the strength of columns. The following is given as supplementary to what is usually found in the works on applied mechanics, and on framed structures, on this subject.

‡ While this is the only purely theoretical column formula which is true in practice, it is only applicable to *very long columns* $\left(\frac{l}{r} > 150 \text{ for pin ends, and } \frac{l}{r} > 200 \text{ for flat ends}\right)$, such as are seldom or never used in actual structures, and hence it is of little practical value. *This formula must never be used for the ordinary lengths.*

r = least radius of gyration of the cross-section of the column, in inches.

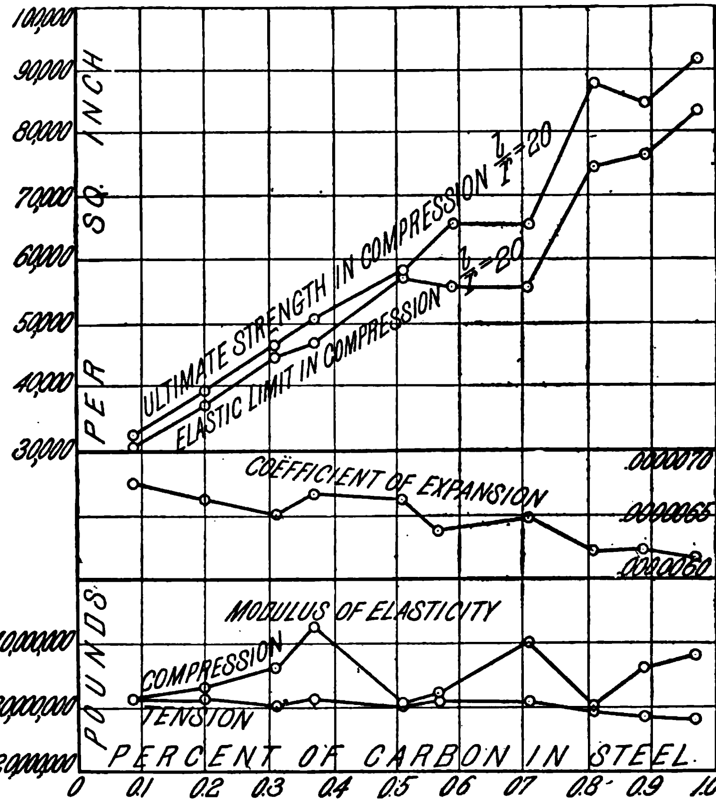

FIG. 294.—Variation of Moduli with increasing Percentage of Carbon in Steel. (*Wat. Ars. Rep.*, 1886.)

In Fig. 296 the locus of this equation is shown for $E = 30,000,000$, the coordinates being p and $\frac{l}{r}$.

Theoretically, for a perfect column, centrally loaded, the strength is constant for increasing lengths, this strength being the "apparent elastic limit" or "yield-point" of the material (*see Fig.* 294) *until the critical length is reached under which the column bends indefinitely under its maximum load, when for any further increase in length the load which will produce this bending regularly diminishes in accordance with the law of Euler's curve.* The theoretical locus, therefore, for the value of p plotted to $\frac{l}{r}$ would be a horizontal line at the apparent elastic limit of the material, extended to an intersection with Euler's curve, and then down along this curve indefinitely. But because no column is perfectly straight, nor, perfectly free to turn, nor

loaded and supported exactly in its gravity axis, nor has the same modulus of elasticity in all its parts, nor is of exactly uniform cross-section, etc., it follows that any locus derived from experiment would usually fall below this theoretical locus, and could never rise above it except from a higher modulus of elasticity, or from a higher elastic limit, or from more fixed end conditions than had been assumed.

In making tests of metal columns there are but two conditions of end supports to which any theory can be adapted, these being *rigidly fixed* in direction and *absolutely free to revolve.* As it is impossible to satisfy the former condition, the latter becomes the only one to which any theoretical formula should be expected to conform. It seems remarkable, therefore, that, so far as the author is aware, *there has been but one set of observations made which has fairly satisfied this requirement*, these having been made by *M. Considère, Ingénieur-en-chef des Ponts et Chaussées*,* France. Both Prof. Bauschinger and Prof. Tetmajer attempted to satisfy this condition, but they mounted their columns with cone or knife-edge bearings at the *computed* gravity axes, while M. Considère mounted his with lateral-screw

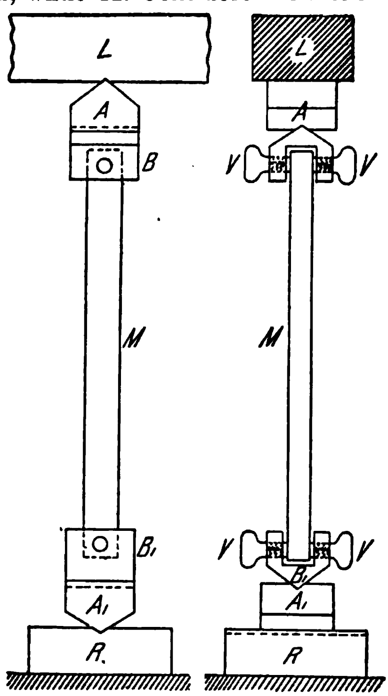

FIG. 295.—Considère's Mounting for Column Tests. (*Fr. Com. Rep.*, vol. III. p. 124.)

adjustments, as shown in Fig. 295, and arranged a very delicate electric contact at the side so as to indicate a lateral deflection as small as 0.001 mm.

* First reported in 1889, and described in vol. I. p. 128 and vol. III. p. 124 of the *Report of the French Commission on the Methods of Testing Engineering Materials*, 1895.

He then applied moderate loads to the columns and adjusted the end-bearings until they stood under such loads rigidly vertical, with no lateral movement whatever.* Then with his double knife-edge bearings at each end, as shown in Fig. 295, the columns were perfectly centred and absolutely free to move or turn about their end bearings, as the theory demands. With these conditions perfected he made 155 tests of columns, of various lengths from $\frac{l}{r} = 40$ to $\frac{l}{r} = 346$, and on various forms of cross-section.

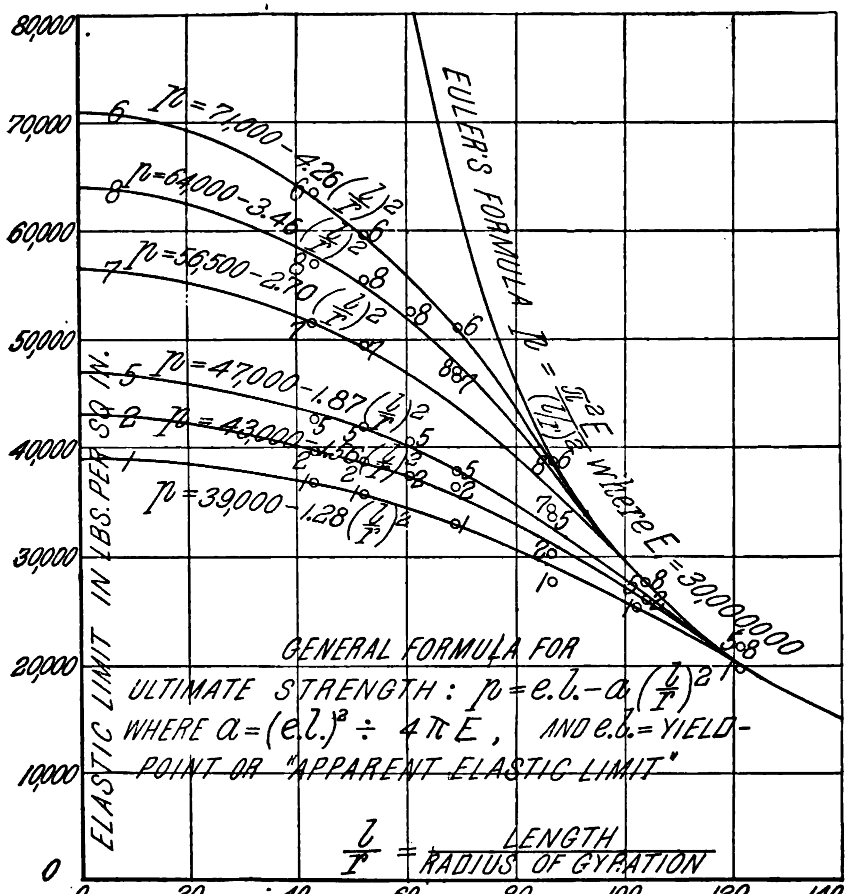

FIG. 296.—One Series of Results of M. Considère's Column Tests, with Material having different "Apparent Elastic Limits." (*Rep. Fr. Com.*, vol. III. p. 124.)

In Fig. 296 the author of this work has plotted the tests made on solid rectangular steel bars, 10 mm. by 17 mm. in cross-section, of six degrees of

* This precaution is essential to a perfect test of the *material* of which the column is composed. Only in this way can other sources of weakness be eliminated. It is to the interest of the contractor, therefore, to provide these appliances.

hardness. He has also fitted to these six sets of observations parabolic loci which cut the axis of loads at the respective "apparent elastic limits," or "yield-points," of the material,* and which are made to become tangent to Euler's theoretical curve drawn for $E = 30,000,000$. The close agreement of these loci with their respective sets of observed ultimate strengths would seem to indicate that they cannot well be improved upon, and that therefore this parabolic law may fairly be assumed to fit the actual facts in an ideal set of experiments as closely as it is possible to do.†

It further appears from these curves that the coefficient of the subtractive term $\left(\frac{l}{r}\right)^2$ follows a very definite law, as shown in "Modern Framed Structures," p. 150. Using this theoretical value of this coefficient, we have, as the maximum strength of any pivoted wrought-iron or steel column, in pounds per square inch for

$$E = 30,000,000, \quad p = \frac{(\text{el. lim.})^2}{4\pi E}\left(\frac{l}{r}\right)^2. \quad . \quad . \quad . \quad . \quad (1)$$

To show that the strength of a column is no function of the ultimate strength of the material either in tension or compression, M. Considère cold-rolled the medium hard steel in No. 5, which had an apparent elastic limit of 47,000 lbs. per square inch, until it had elongated ten per cent of its original length. This raised its elastic limit to 71,000 lbs. per square inch, while its ultimate tensile strength was raised only from 83,000 to 88,500 lbs. per square inch. Thus metal No. 6, with an elastic limit of 71,000 lbs. and an ultimate strength of 88,500 lbs. per square inch, was over 10 per cent stronger in columns than metal No. 8, which had an elastic limit of 64,000 lbs. and an ultimate strength of 98,000 lbs. per square inch.

While the parabolic curves here given are purely empirical in form, theory dictates:

1. That this locus shall start horizontally from the vertical axis at the "apparent elastic limit";
2. That it shall become tangent to Euler's curve; and
3. That it shall have no points of inflection other than the point of tangency with Euler's curve.

While there may be an infinite number of curves which would satisfy these requirements, the parabola is the simplest of all, and it also seems to fit the observations as well as any, whether these observations be made under ideal conditions, as in Fig. 296, or under the nearly ideal conditions of Prof. von Tetmajer, Figs. 297 and 298, or under the conditions of practice, as in Fig. 208 of "Modern Framed Structures."

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* As computed from the ultimate strengths which alone were given in the original communication, the yield-points not having been observed.

† The author had already developed this curve as best representing the strength of ordinary columns too short for Euler's formula to apply to, in "Modern Framed Structures," p. 148 (1892).

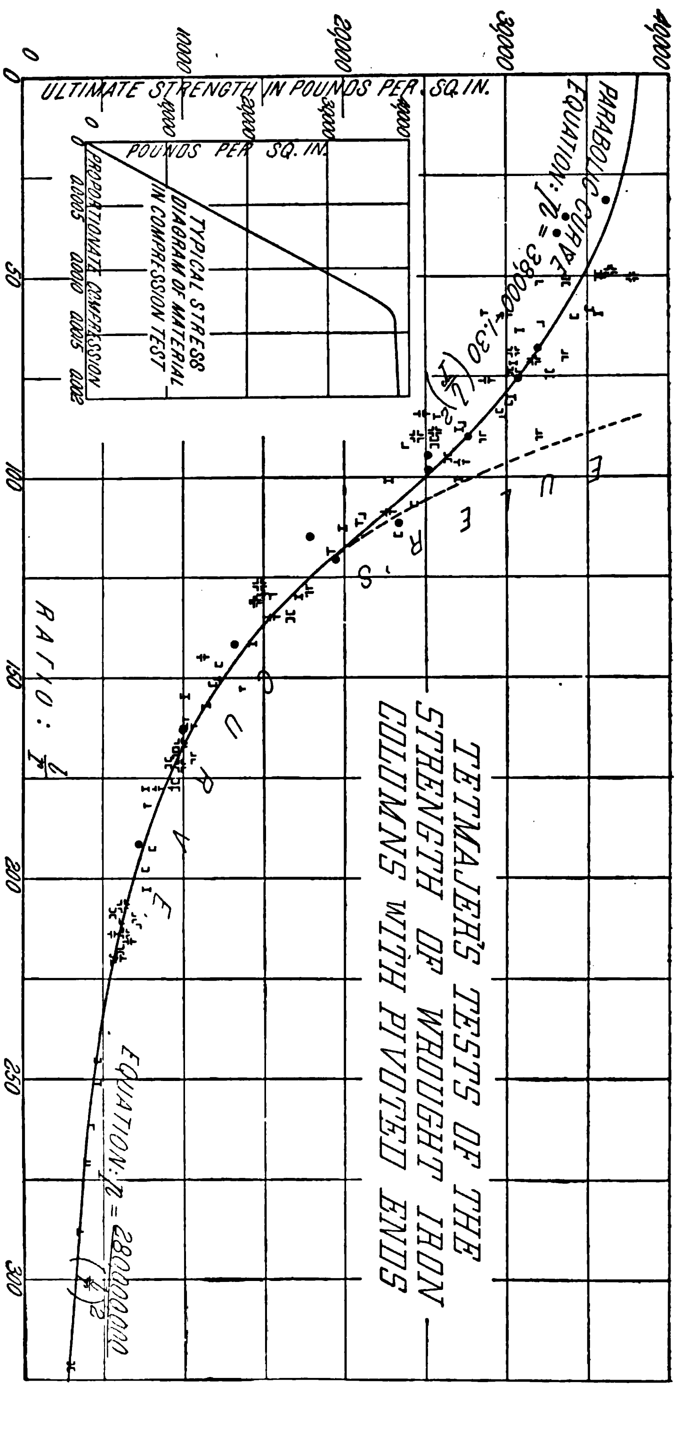


Fig. 297.—The Author's Parabolic Column Formula fitted to Tetmajer's Tests on Wrought-iron Columns. (*Communications*, vol. IV.)

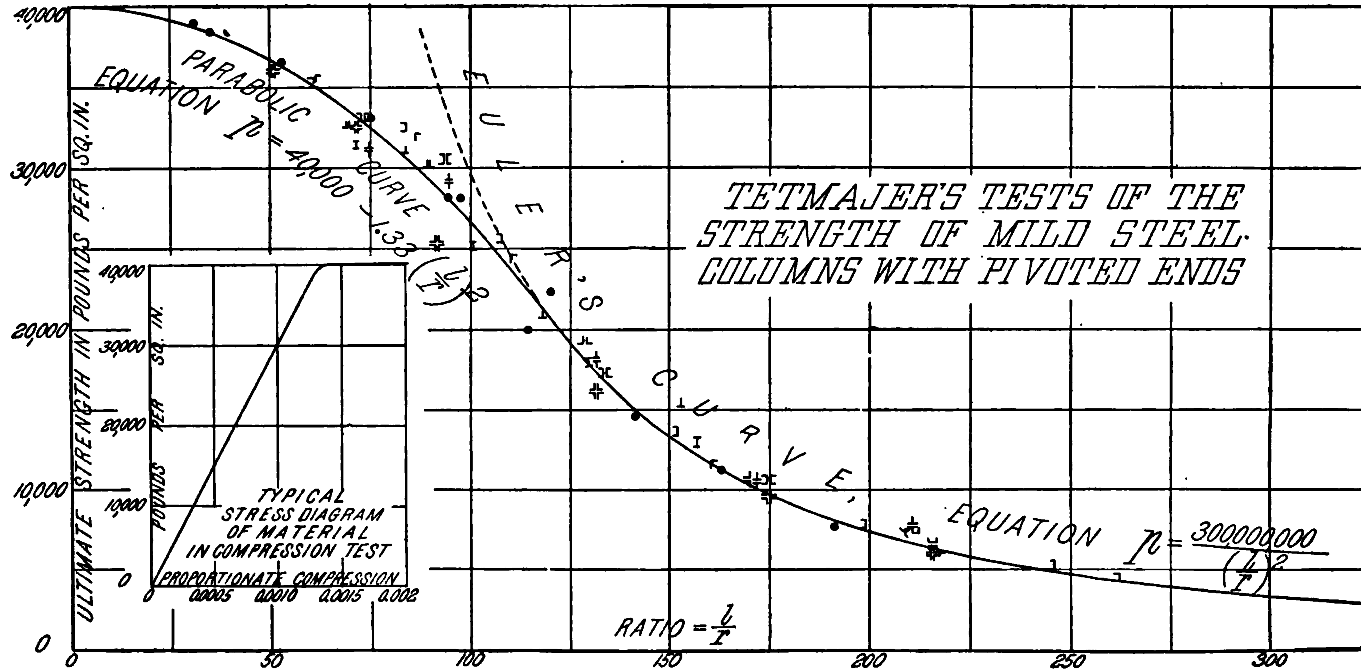


FIG. 298.—The Author's Parabolic Column Formula fitted to Tetmajer's Tests of Steel Columns. (*Communications*, vol. IV.)

Prof. von Tetmajer's tests cover a great variety of forms, simple and composite, on both wrought-iron and steel, and the results of these tests are all plotted, with characteristic symbols, in Figs. 297 and 298. While these results scatter somewhat, owing to varying elastic limits of the specimens and to the fact that the knife-edge bearings were placed at the computed gravity axes, and were not adjusted to the true centres by lateral adjustment under small loads, as were those of M. Considère, still the parabolic curve fits the average position of the plotted points as well as could be desired. See also the similar diagram for wooden columns in Chapter XXXII.

The following are the author's parabolic column formulæ as given in "Modern Framed Structures":

ULTIMATE STRENGTH OF COLUMNS, IN POUNDS PER SQUARE INCH.

For Wrought-iron Columns, Pin Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 170,\right)$

$$p = 34{,}000 - .67\left(\frac{l}{r}\right)^2. \qquad (2)$$

For Wrought-iron Columns, Flat Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 210,\right)$

$$p = 34{,}000 - .43\left(\frac{l}{r}\right)^2. \qquad (3)$$

For Mild-steel Columns, Pin Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 150,\right)$

$$p = 42{,}000 - .97\left(\frac{l}{r}\right)^2. \qquad (4)$$

For Mild-steel Columns, Flat Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 190,\right)$

$$p = 42{,}000 - .62\left(\frac{l}{r}\right)^2. \qquad (5)$$

For Cast-iron Columns, Round Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 70\right)$

$$p = 60{,}000 - \frac{25}{4}\left(\frac{l}{r}\right)^2. \qquad (6)$$

For Cast-iron Columns, Flat Ends, $\left(\frac{l}{r} \overline{\underline{<}}\, 120,\right)$

$$p = 60{,}000 - \frac{9}{4}\left(\frac{l}{r}\right)^2. \qquad (7)$$

For White-pine Columns, Flat Ends, $\left(\frac{l}{d} \leqq 60,\right)$

$$p = 2500 - .6\left(\frac{l}{d}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

For Short-leaf Yellow-pine Columns, Flat Ends, $\left(\frac{l}{d} \leqq 60,\right)$

$$p = 3300 - .7\left(\frac{l}{d}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

For Long-leaf Yellow-pine Columns, Flat Ends, $\left(\frac{l}{d} \leqq 60,\right)$

$$p = 4000 - .8\left(\frac{l}{d}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

For White-oak Columns, Flat Ends, $\left(\frac{l}{d} \leqq 60,\right)$

$$p = 3500 - .8\left(\frac{l}{d}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

To obtain from the above working formulæ for designing, divide both terms of the right-hand members of these equations by the factor of safety chosen for the work in hand and for the material used. The smallest factors would be used with rolled mild steel, and the largest with timber and cast iron.

285. Spring Testing-machines.—Fig. 299 shows a form of spring testing-machine adapted for both compression and tension tests, the former being made at *A* and the latter at *B*. It is made in two sizes, of 2500 and 4000 lbs. capacity respectively.

In Fig. 300 is shown a spring testing-machine of 65,000 lbs. capacity, for compression only, and not requiring the use of over-weights, although such are furnished for one half the total load, if desired.

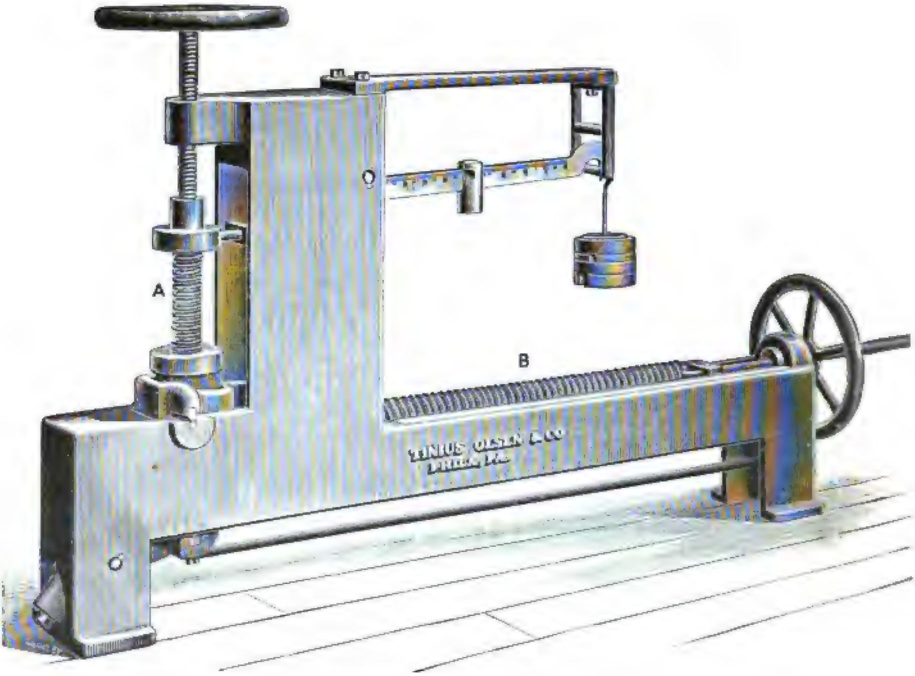


FIG. 299.—Spring Testing-machine for both Tension and Compression Tests.

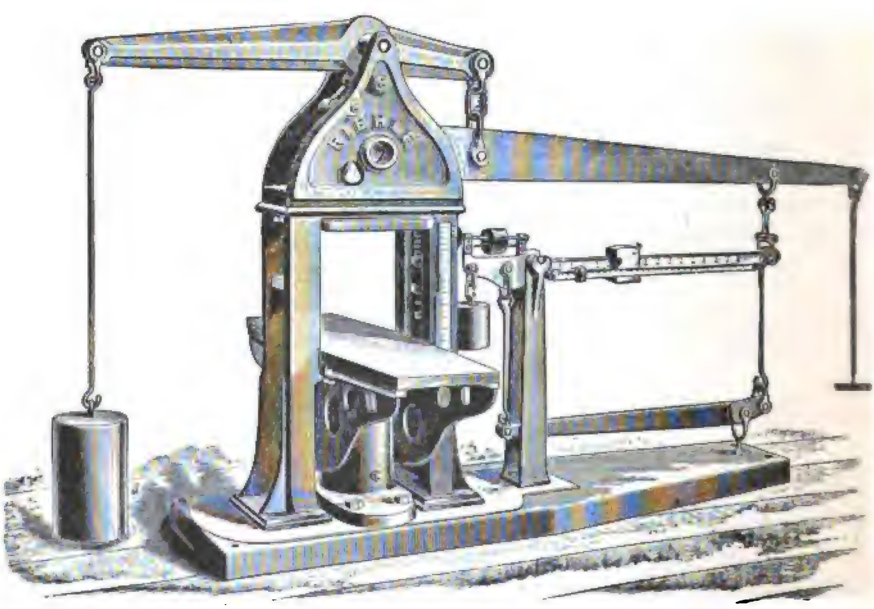


FIG. 300.—Spring Testing-machine.

CHAPTER XVII.

CROSS-BENDING TESTS.

286. Objects of Cross-bending Tests.—Brittle materials like cast-iron, stone, brick, and concrete are tested in cross-bending to determine their ultimate strength, and perhaps also their resilience. Timber is so tested also to determine its ultimate strength and its modulus of elasticity. Springs and spring-steel are tested in this way to obtain their elastic limits and their deflections under given loads, and railroad rails are sometimes tested for elastic limit and ultimate strength. Cross-bending tests are also made for scientific purposes to test the correctness of the ordinary formulæ for the strength and the deflection of beams.

Since three kinds of stress, tension, compression, and shearing, are developed when a beam is bent under the action of external forces, the problem is more complex than those considered in the two previous chapters Usually the shearing stresses are left out of account in designing both for strength and stiffness, but the conditions under which this stress should be recognized and taken account of are given in Article 38, for strength, and Article 46, for deflection.

287. Essential Considerations in Cross-bending Tests.—The essential conditions which must be satisfied in making cross-bending tests are:

1. The loads should be applied centrally in the direction of the greatest or of the least moment of inertia of the beam, in order to prevent torsion.

2. The supports must be rounded knife-edges, bearing on auxiliary plates if necessary to prevent indentation.

3. The loads should be continuously progressive, without shock, and in the case of timber they must increase at a fixed rate with no stopping when readings are taken.

4. The deflections must be measured by observing the movement of the neutral plane at the loaded point with reference to the neutral plane at the two end supports. That is to say, the deflection apparatus must be attached directly to the specimen, or rest on the end bearings, and be self-contained with the test specimen, and independent of all deforming movements of the machine itself.

The fourth condition is seldom properly satisfied. It is common to measure deflections with reference to some part of the frame of the testing-

machine, assuming this to be rigid, or by means of a deflection apparatus attached to this framework and moving with it.

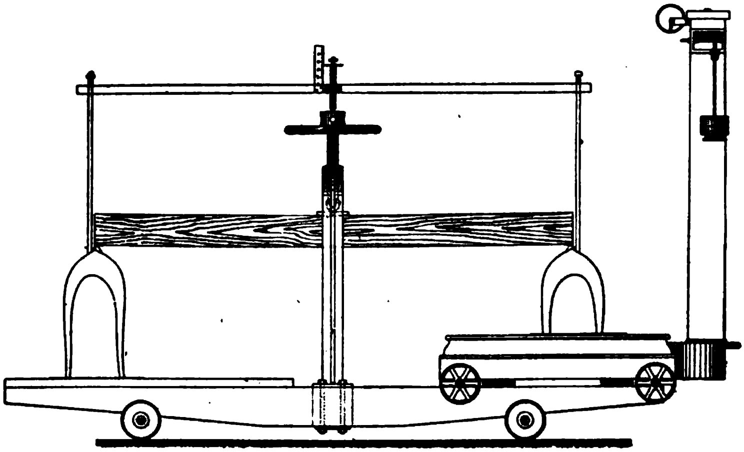


FIG. 301.—The Author's Beam-testing Machine.

Cross-bending Testing-machines.—In the author's machine, shown in Fig. 301, the deflection is measured by means of a micrometer-screw, reading to 0.001, inch held in place by a bar which is attached directly to the knife-edge bearings *through parts which are not under stress.* The micrometer-screw bears upon the top of the power screw which presses on the centre knife-edge. As all these bearings have steel plates intervening between them and the specimen (if this be timber), the movement of the centre bearing, with reference to the end bearings, is registered on the micrometer-screw, and this is the deflection of the specimen. One half the load is weighed on any ordinary form of platform-scales.

In the large beam-testing machine of the author's, shown in Fig. 302, used mostly for testing large wooden beams, the deflection is measured by means of a fine thread attached (at one end by a rubber band) to two nails driven into the stick in the neutral plane over the end supports. At the centre a nickel-plated scale, graduated to 0.1 in. and polished to act as a mirror, is fastened to one or both sides of the beam. The thread is then read on this scale by bringing it and its image into coincidence and estimating its position to the nearest 0.01 in. The load is applied by pumping oil into the cylinder below, thus depressing the screws and the cross-head carried by them, and one half the load is weighed on the 50,000-lb. platform-scales under one end. The base of this machine consists of two long-leaf yellow-pine sticks, 6 × 18 inches in section and 24 feet long, with a $\frac{3}{4}$×18-inch steel plate inserted between them. Its capacity is 100,000 lbs.

In the machine shown in Fig. 303, specially designed for cast-iron tests, the deflection is correctly indicated on the graduated arc by means of an

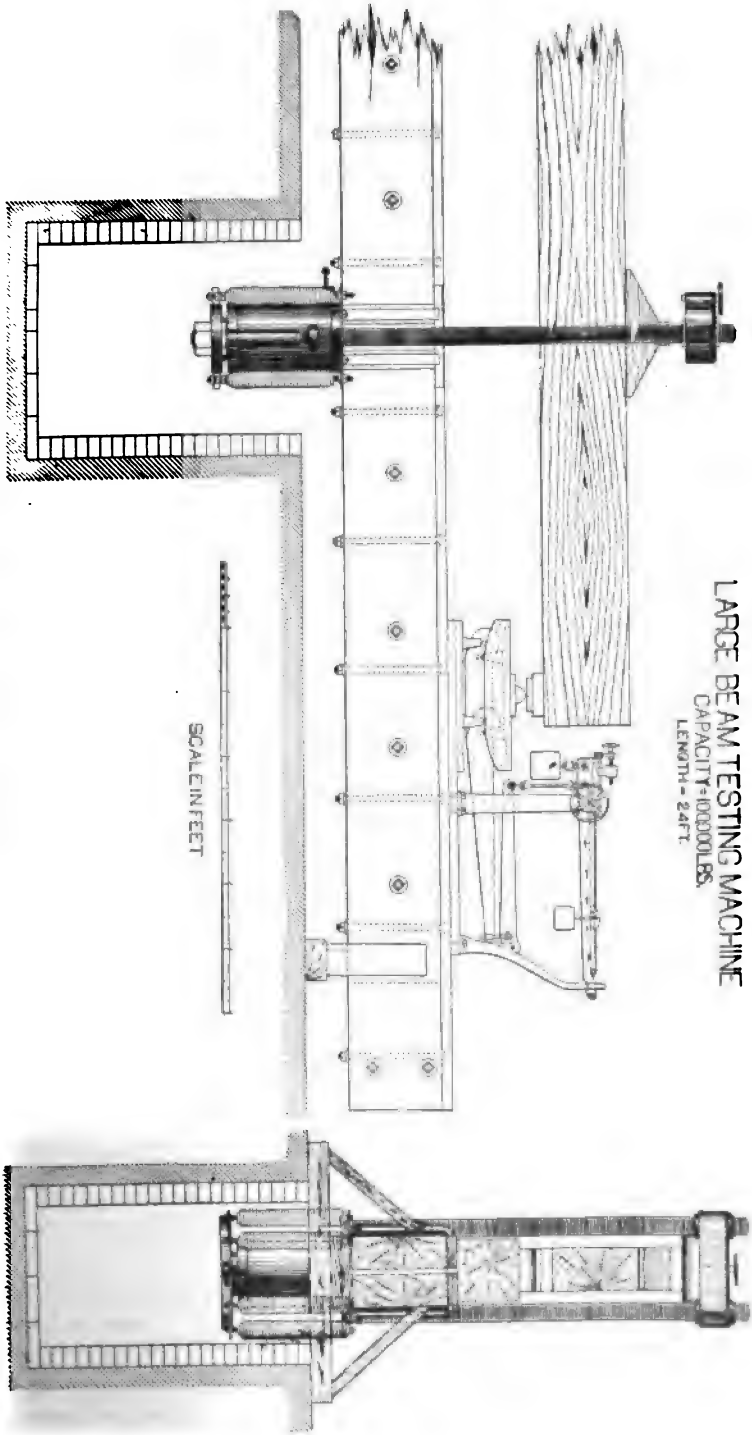


Fig. 302.—Large Beam-testing Machine designed and used by the Author.

ingenious arrangement of levers underneath, not shown in the figure. Its capacity is 4000 lbs.

In Fig. 304 is shown Keep's autographic recording transverse test appa-

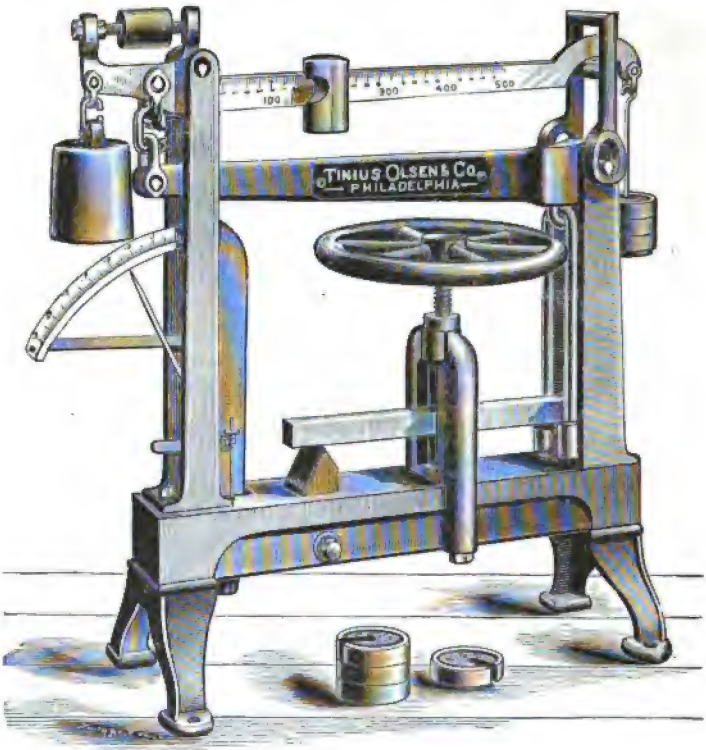


FIG 303.—Cross-bending Testing-machine for Cast Iron. Deflection correctly measured.

ratus for his standard form of specimen ½ inch square by 12 inches long. It makes an autographic record like those shown in Chapter XXIV, the deflection of the test specimen being taken up by the gradual falling of the weighing-beam. The movement of the poise also moves the paper, while the deflection of the specimen moves the pencil. This is a valuable machine for tests on this size of specimen.

The universal machines shown in Figs. 256 to 260, and in Figs. 266 and 270, are all adapted to making transverse tests by inserting an I beam or other rigid base on the weighing-table when the specimen is longer than this, and supporting the specimen on it.

288. Importance of Measuring the Deflection in Transverse Tests of Cast Iron.—The importance of measuring the deflection of transverse-test specimens of cast iron, as well as the breaking strength, is now generally recognized. The resistance of the metal to shock is measured by the product of

the ultimate load into the final deflection, divided by 2, this being approximately the total area of the stress-diagram in cross-bending. If this be now divided by the volume of the metal between the end bearings, it gives resistance to shock in inch-pounds per cubic inch of metal.* Since it is more convenient, however, to weigh the bar than to compute its volume, the resistance to shock is commonly computed per pound of metal (between end bearings).

From many experiments made by the author, he has recommended the following requirements † for test-bars about 1 inch square:

	Inch-pounds per pound of Cast Iron.
For the lower grades of castings....	20–30
For good machine castings.........	40–50
For stove-castings, and for impact machinery......................	60–70

In this way both the strength and the deflection are properly allowed for; and since these results usually vary inversely with each other, both may vary greatly without showing an appreciable variation in this product, and hence without appreciably changing the value of the metal. On the other hand, the strength may be very high, with a very small resistance to shock; that is, it may be strong in a static test, but very brittle. These products will be greater for small (thin) specimens than for thicker ones; so the only safe rule is to find by trial what products can be expected and demanded for any given product and size of specimen.

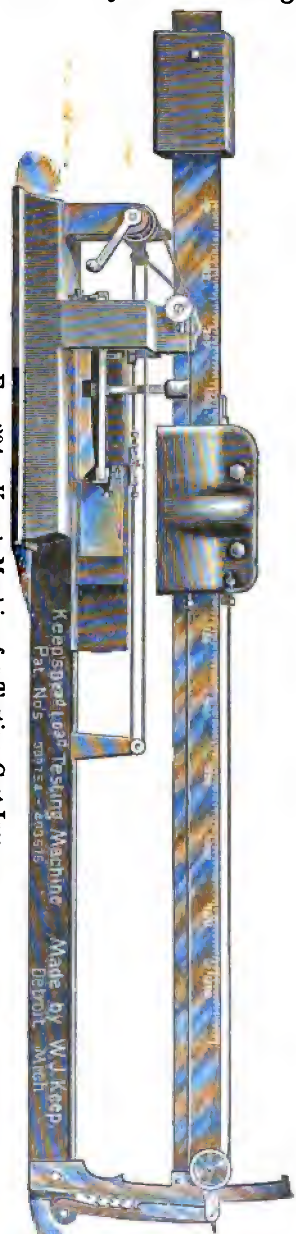


FIG. 304.—Keep's Machine for Testing Cast Iron.

289. The Computed Strength in Pounds per Square Inch on the external fibres of a transverse-test specimen is found from the formulæ given in Art. 33. Thus for any form of section tested to failure by a load at the centre the "modulus of rupture" in cross-breaking is

* It was shown in Arts. 55 to 57 that the resilience was always proportional to the *volume* of the body subject to stress, it being independent of the dimensions of the body so long as the form of cross-section remained the same.

† See a paper by the author on *Cast Iron*, Trans. Am. Soc. C. E., vol. XXII. p. 91.

$$f_r = \frac{My_1}{I}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

while for a solid rectangular cross-section

$$f_r = \frac{3\,Wl}{2bh^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and for any form of cross-section we should have

$$f_r = \frac{Wly_1}{4I}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where f_r = modulus of rupture in pounds per square inch;
W = breaking-load at centre in pounds;
l = length between end bearings in inches;
b = breadth in inches;
h = height in inches;
y_1 = distance from neutral axis to outside fibre which failed under the breaking-load, in inches;
I = moment of inertia of the cross-section about its neutral axis;
M = bending moment on the beam at the section of rupture.

Because these formulæ are strictly true inside the elastic limit, it must not be inferred that this so-called "modulus of rupture in cross-bearing" represents any actual stress on any outside fibre, either in tension or compression, at the time of rupture. (See a discussion of this question in Arts. 35 and 36, p. 50.) In general this modulus is about twice the tensile strength, in the case of cast iron, while in timber it is somewhat below an average of the tensile and the compressive strength of the wood. With this understanding this modulus is a convenient, though conventional, method of stating the strength of any material under cross-breaking stress, and for comparative purposes it is very useful. For the purposes of the designer, however, these formulæ are strictly correct, since he always works with loads and stresses inside the elastic limits of the materials he uses.

290. The Modulus of Elasticity (Stiffness) is preferably found from a transverse test, since it is mostly used for computing the deflection of beams. Since this quantity is of necessity computed from loads and their corresponding deflections inside the elastic limit, it follows that this modulus is found to be practically the same whether it is computed from tensile, compressive, or transverse tests. Thus Prof. Tetmajer tested fourteen rolled wrought-iron I beams, from 4 to 16 inches in depth, and obtained from them an average value of $E = 27{,}840{,}000$, while on thirty-one tension tests on specimens cut from shape iron from the same metal he obtained a value of $E = 28{,}110{,}000$; for wrought-iron riveted plate girders, from 16 to 28 inches deep, he obtained a value of $E = 26{,}000{,}000$.

In mild steel he found

For the tension specimens $E = 30{,}550{,}000$;
" " riveted plate girders $E = 28{,}160{,}000$.

CHAPTER XVIII.

IMPACT AND HARDNESS TESTS.

IMPACT TESTS.

291. Object of Impact Tests.—As explained in Art. 53, impact tests cannot give absolute results, like those obtained from tension, compression, and transverse tests, and hence they are properly used only where other methods of testing are not available. They are commonly employed on cast-iron car-wheels, on cast-steel and malleable-iron car-couplers, and on car-axles and sometimes on rails and rail-joints. Axles and rails, however, can be tested statically in cross-bending, and more can be learned by testing them in this manner, by bending them back and forth, and plotting their bending-stress diagrams, than by the drop tests.*

Because of the extreme difficulty of arranging an impact test so as to give to the specimen a plain tensile stress, without allowing a large and uncertain part of the energy of the blow to be absorbed in the auxiliary appliances, this has seldom been attempted, and certainly it never has succeeded in giving any valuable results.

Impact tests in compression are seldom employed except to produce penetration of a standard form, to determine hardness, which will be described later under the head of *hardness tests.* The impact test given to car-couplers might be called a compression test, perhaps, since the blow is given "end-on"; but as failure here occurs by breaking off portions of the enlarged head, by developing in it excessive transverse stresses, it is really a transverse test.

In general, therefore, impact tests are all transverse, or cross-bending, tests.

The author has also introduced a species of impact test for street-paving brick in place of the abrasion test hitherto employed. This was done from the fact that paving-brick do not wear out by abrasion, but by being broken down by the blows from horse's shoes, and from the wheels of vehicles.

* An exception may have to be made in the matter of brittleness of metals induced by very low temperatures, which can, it is said, only be determined by impact or drop tests.

Impact tests, therefore, are usually made to determine the resistance to shock of structural forms which cannot readily be tested in any other way.

292. Essential Conditions of Impact Tests.—Since the force of a blow depends as much on the resistance offered by the body struck as it does on the striking body, it follows that the anvil, or bed, of an impact machine is quite as important as the weight of the ram and the height of its fall. A standard impact test, therefore, involves a standard size of anvil and a standard froundation for it, quite as much as a standard weight of hammer and standard fall of same.

The pendulum machine would seem to offer one advantage, however, which cannot be realized in drop machines. The pendulum machine can be so designed as to allow the pendulum weight to pass the specimen when it breaks, and by automatically recording its extreme movement, and deducting this vertical component from the original total fall, the actual energy absorbed by the specimen, up to rupture, would be determined proivded the anvil is rigid.* In this way the specimen could be broken on the first blow, and the energy spent upon it (and absorbed by it) exactly determined. This would seem to be the only proper way to make comparable impact tests.

The Pennsylvania Railroad Company has standardized the impact test of cast-iron car-wheels and of car-axles, and the American National Car-builders' Association has now (1896) standardized the test for car-axles as indicated in Art. 294; but with these exceptions it can hardly be said that impact tests made in different places in this country can be considered as at all comparable, because of a want of identity in the foundation portion. If the impact machine be of the pendulum form, it must strike the specimen at the centre of percussion of the entire pendulum in order to prevent a portion of the energy from spending itself by bending the pendulum. While pendulum machines are more convenient, drop machines are more certain to deliver the full theoretical force of the blow. In a pendulum machine the energy of the blow is, of course, the total weight of the pendulum into the distance through which its centre of gravity falls.

293. The Energy of the Blow.—The unit of measure in impact tests is the foot-pound (or kilogram-meter). This energy cannot be measured in pounds, and *no scheme of equivalents can be devised between the foot-pound units of an impact test and the pound units of a static test*, although this has often been attempted. There is no relation between the resistance to shock and the resistance to a static load, since there is no relation between the total area of a stress-diagram and its stress coordinate. The attempt which is often made, therefore, to equate these two kinds of resistance is as foolish as the ancient practice of estimating the discharge of a stream, or aqueduct, or pipe from its cross-section alone.

From the law of the conservation of energy we have:

* Some experiments along this line have recently been made by Mr. S. B. Russell, M. Am. Soc. C. E., in the St. Louis Water-works Department.

The work which gravity does on the falling weight, and which is wholly represented by the energy of the hammer at the time it strikes, must be absorbed by the resisting body. This energy is equal, in foot-pounds, to the weight of the ram in pounds multiplied by its total vertical fall (including the vertical deflection of the specimen) in feet.

In the case of a pendulum impact machine the entire weight of the swinging parts must be divided into two parts, and these parts concentrated at the axis of rotation and at the centre of percussion. The latter part, only, multiplied by its vertical drop is the measure of the energy of the blow (but this is the same as the total weight into the fall of its centre of gravity).

To find the centre of percussion and the equivalent weight to be considered as concentrated at this point, a graphical solution may be employed, as follows:

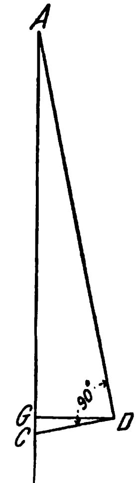


FIG. 305. — Graphical Method of Finding the Centre of Percussion.

Let AG extended, Fig. 305, be the pendulum, with its axis of rotation at A. Let G be the centre of gravity of the entire pendulum, with all its rigidly connected parts (to be found by trial). Let GD, drawn perpendicular to AG at G, be made (to the given scale) equal to the radius of gyration of the entire pendulum about its centre of gravity G, (to be computed).

Then draw AD, and DC perpendicular to AD, cutting AG extended in C. Then is C the centre of percussion of the pendulum.* If the graduated arc (see Fig. 308) have a radius equal to AC, and the vertical components of the pendulum's motion (versed sines) be laid off on this arc, then the equivalent weight to be concentrated at C is to be used for computing the energy of the blows, and this equivalent weight is

$$W_c : W :: \overline{AG} : \overline{AC};$$

or equivalent weight at

$$C = W_c = \frac{W . \overline{AG}}{\overline{AC}}; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where W is the total weight of the pendulum used in finding the centre of gravity G.

If the graduated arc has a radius equal to AG, then the total weight W is to be used with the versed sine to compute the energy of the blow.

If a conventional radius, R, has been used by the maker of the machine, then a corresponding W_r must be employed which will satisfy the equation

$$W_r = \frac{W\overline{AG}}{R}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

In every case, however, the point of impact of the pendulum should be at C, the centre of percussion.

* Rankine's Applied Mechanics, Art. 581.

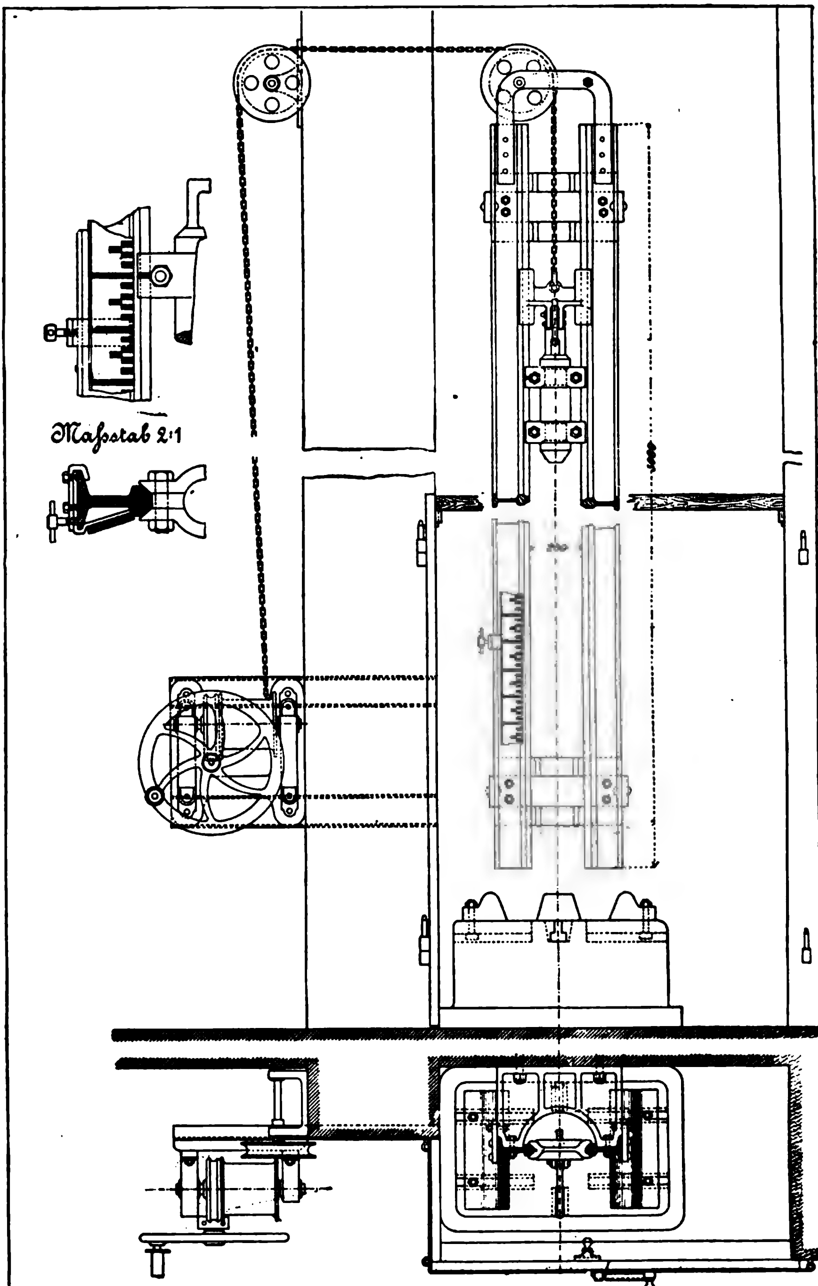


FIG. 306.—Most Approved Form of Impact-testing Apparatus. (*Rep. French Commission.*)

294. Impact-testing Machines.—In Fig. 306 is shown the impact-machine designed and used by Prof. A. Martens in his testing laboratory at Charlottenburg, Germany.* Its dimensions are given in meters. It admits of an extreme fall of 4.5 meters (about 15 feet), and of a weight of ram of 200 kilograms (440 lbs.), although he has used weights of 36 and 56 kilograms only. It is intended for specimen tests only. The anvil weighs 1250 kilograms (2750 lbs.), or 22.5 times the heaviest ram usually employed. This in turn is set on a strong cement-masonry foundation, separate from that of the building,† as should always be done with drop machines.

The National Car-builders' Association of America has (1896) adopted a spring support to the anvil, as shown in Fig. 307, in order to insure perfect

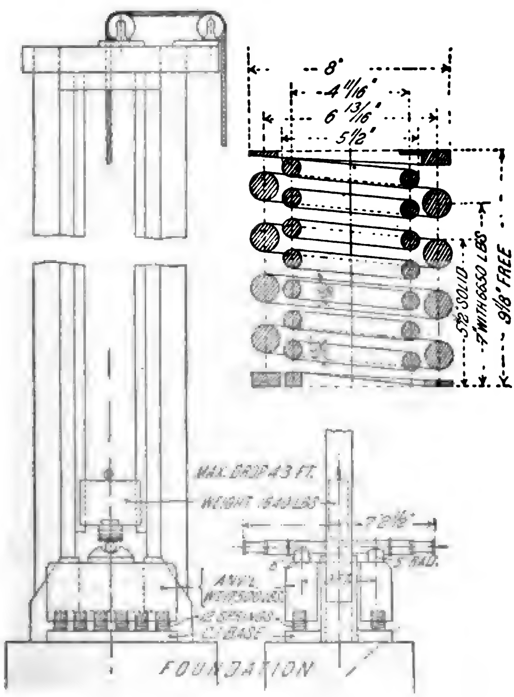


FIG. 307.—Standard Impact-testing Machine with the Anvil-block on Springs, as recommended by the *Master Car-builders of America*, 1896. Previously used by the Penn. Ry. Co.

identity of reactions in different machines. This is perhaps the only way to eliminate the varying effects of different foundations of the anvil-block.

Fig. 308 gives a view of Keep's autographic recording pendulum impact-

* See *Mittheilungen aus den Königlichen Technischen Versuchsanstalten zu Berlin*, 1891, p. 2, and Plate I. The machine was made by E. Becker, machinist, Berlin.

† At first it was set on the floor, but it was found necessary to put it on a more solid foundation.

machine. It is used only for testing his standard cast-iron bars $\frac{1}{2}$ inch square and 12 inches long. The hammer weighs 25 lbs., and swings on a

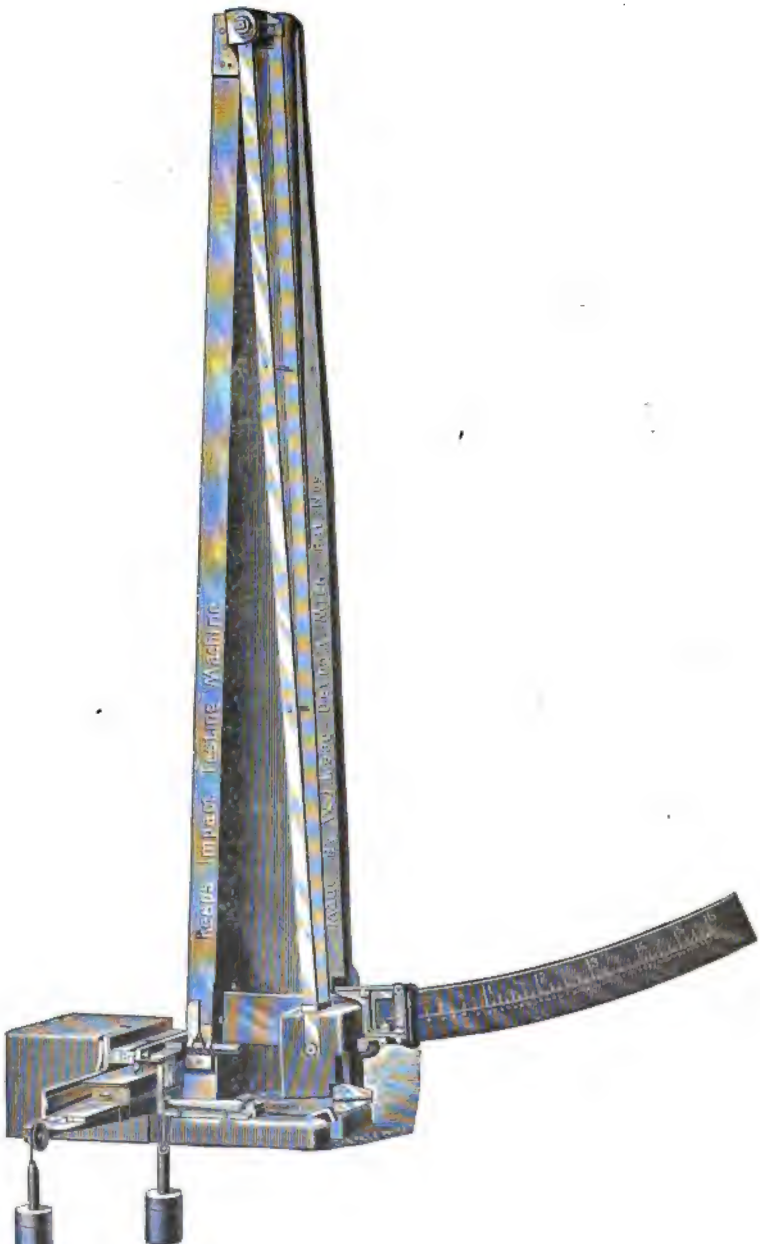


FIG. 308.—Keep's Impact-testing Machine.

radius of 6 feet. The weight of the anvil is admitted to be too light, and the designer recommends that it be set against a brick wall! * The arc is graduated to vertical drops of $\frac{1}{8}$ inch, with a total fall of 6 inches. The paper on which the deflection is recorded at each blow is automatically moved $\frac{3}{16}$ inch after each blow, so that the record consists of a series of parallel lines, each being the deflection for that blow, magnified four or five times by the leverage of the recording apparatus.

TESTS FOR HARDNESS.

295. Hardness Defined.—The term *hardness* is used in two senses, as applied to metals, minerals, and other solids. It is used to signify—

(*a*) *Resistance to indentation* (permanency of form);

(*b*) *Resistance to abrasion or scratching* (permanency of substance).†

These two kinds of hardness are more or less related, and are often confused. In practice the demands for these two kinds of hardness are quite distinct, and hence two very distinct kinds of tests are employed to determine them.

296. Hardness Test for Permanency of Form or Resistance to Indentation.—The only test of this kind which has ever been standardized is the indentation test by means of a pyramidal steel punch, attached to a falling

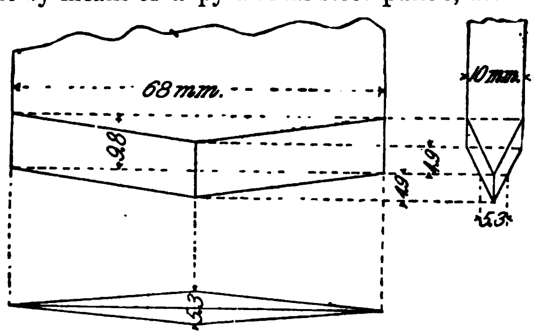


FIG. 309.—The Rodman Steel Punch for Hardness Tests. Dimensions in millimeters. (*Rep. Fr. Com.*, vol. III. p. 261.)

weight. The form most favorable to exact results is that chosen by Lieut.-Col. T. J. Rodman (U. S. A.) before 1860,‡ and shown in Fig. 309 in metric measurements. These are used because this test has been standardized in France, and the *degree of hardness* is given in metric units.§ Such a steel point is rigidly attached to the base (or striking side) of the ram in an impact-testing machine, such as shown in Fig. 306 or 307 or 308. The sur-

* Evidently the particular character of this setting will greatly affect the force of the blow on the specimen.

† After Osmond.

‡ See his *Report of Experiments on Metals for Cannon and Cannon-powder*, 1861.

§ While the author of this work has, as a rule, expressed quantities in English units only, he makes an exception in this case.

face of the substance to be tested is planed or filed flat and polished. The steel point is then made to fall normally upon the surface from any desired height, and the observation consists in noting

The weight of the ram = W in kilograms;
The height of the fall = h in millimeters;
The length of the indentation = l in millimeters.

The work done upon the body tested, or in producing the indentation, will be Wh kilogram-millimeters, *provided the anvil, or body struck, was very massive and firm in comparison with the weight of the ram.* This is, of course, essential to the correctness of the assumption that the energy of the falling body spends itself wholly in producing the indentation; and this must be assumed and secured for a perfect accordance of results.

The volume of displaced material resulting from the indentation will be ml^3, where m will vary with different forms of pyramids, but will be a constant for any one pyramid, or punch. It has been shown most conclusively by Lieut.-Col. Martel * that when the essential conditions of the test are satisfied,

For all forms of pyramids, for all weights of ram, and for all heights of fall, the volume of the displaced material of a given quality is equal to the energy of the blow (Wh) divided by a constant, D,† which constant is the work or energy necessary to displace (by deformation) a unit-volume of that material. This constant is therefore characteristic of that material and may be taken as its index of hardness, or of its resistance to indentation.

Since the kilogram-millimeter units have already been used in France, and the hardness of many kinds of materials has been found and published on this scale, it would lead to unnecessary confusion to change the units, since this would change the numerical index of hardness.

To find the volume of the pyramidal displacement of the Rodman punch (Fig. 309) from the measured length, multiply the cube of the length by 0.0009413 (log $\bar{4}.97375$), or vol. = $0.0009413l^3$.

For any other form of punch the volume would be readily computed, but approximately this form is best, because it gives a very large length to be measured for a very small volume. In other words, we argue from a longer base, thus making the percentage error of observation correspondingly less. Furthermore, the indentation is shallow and hence injures the material (which may be a finished or unfinished final form) to a less degree. Any other form would, however, give strictly comparable results, so that there is no real necessity of adopting this particular form. It goes without saying that the punch should itself be so hard as not to suffer any permanent deformation in service.

The truth of the theory that $V = \frac{Wh}{D}$ has been established by Martel

* *Commission des Méthodes d'Essai des Matériaux de Construction*, vol. III. p. 261.
† For *dureté*, hardness.

within the limits of the errors of observation, and hence can be accepted with confidence, as giving an absolute standard by which to measure hardness when this implies resistance to indentation. The following table contains values of *D* (degrees of hardness on the Martel scale) in kilogram-millimeter units for various metals.

DEGREES OF HARDNESS ON THE MARTEL SCALE.

Metals Tested.	Degree of Hardness. Kilogram-millimeter Units.*
High carbon ("diamond") steel, hardened in oil..	613
" " " " not hardened....	460
Medium steel (for cannons), hardened in oil......	455 to 300
Hoop-steel (for large guns), hardened in water....	330 to 295
Rolled wrought iron...........................	226
Hammered wrought iron.........................	238
Cast iron (for guns)...........................	300 to 208
Bronze (cast in shells).........................	154
" after cold-hammering...................	238
" after drawing down 12% on a mandrel.....	310
" cast in sand (C 88, Sn 12, Z 2)...........	137
" " " " without zinc................	115
Copper, rolled	156
" reheated and cooled in water............	64
Zinc, rolled..................................	77
Tin, cast.....................................	33
Lead, cast....................................	9

297. Hardness Test for Permanency of Substance or Resistance to Abrasion.—While a great many tests of this property have been devised and used, none of them has given such satisfactory measurable results as the one just described for resistance to induction. The scratch test has long been in use for classifying minerals as to their hardness, and ten grades of hardness are recognized under this test. It is purely relative, and is entirely inadequate to the requirements of the user of metals. By this test the body *A* is harder than *B* when a point or sharp corner or edge of *A* will scratch the surface of *B*, and when the converse will not hold.

Mr. Thomas Turner has devised the instrument shown in Fig. 310, and this has been largely used by both Turner and W. J. Keep for the grading of cast irons for hardness. In this a diamond point is fixed at the base of a vertical pencil which is carried by a perfectly balanced arm. Provision is made for loading the pencil by weights (in grams), and the hardness is indicated by the number of grams required to make a standard scratch on the surface tested. Evidently the standardizing of the scratch offers great

* To change these figures to pound-inch units, multiply by 1422.

difficulties, so that results obtained by this instrument in the hands of different persons would probably not be strictly comparable.

Prof. Martens (Berlin) has undertaken to standardize this instrument by making the load on the pencil a constant and measuring the width of the

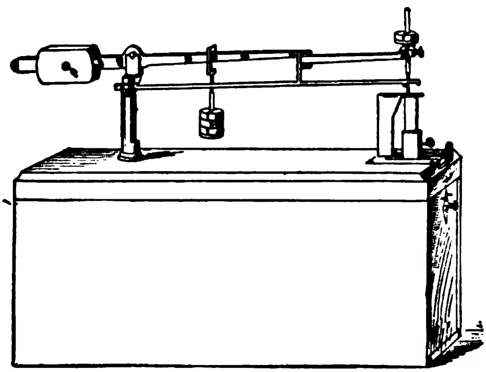


FIG. 310.—Turner's Apparatus for Testing Hardness.

scratch with a micrometer-microscope, and this is the method employed by the German artillery officers.

Standard abrasion-machines have also been used (see Fig. 288), but it is almost impossible to duplicate the conditions exactly.

In general, therefore, it may be said that there is now no absolute test for hardness as meaning resistance to scratching or abrasion. (For a brief account, without illustrations, by Osmond, of the many devices which have been tried, see *Report of the French Commission*, vol. III. p. 279.)

CHAPTER XIX.

SHEARING AND TORSION TESTS.

SHEARING TESTS.

298. Essential Conditions of a Shearing Test.—In order to obtain the true shearing strength of any substance it is necessary to develop in it, along a given plane, shearing stress only, unaccompanied by the bending stresses of tension and compression. To accomplish this it is necessary to concentrate the external forces of action and reaction on planes an infinitely small distance (dx) apart. Any finite distance between these planes will develop a cross-bending action and its resultant direct stresses across the plane of shear. As it is impossible to so concentrate the external shearing forces, it is necessary to overcome the bending stresses set up by the non-concurrence of the external forces *by preventing the bending of the specimen* subjected to these forces. This can only be done by reinforcing the specimen between the shearing planes. This may be done by grooving the specimen in the planes of shear, or by supporting it by auxiliary clamps. As neither of these expedients has usually been resorted to in shearing tests, it follows that very few such tests have ever been made in which shearing stress has been unaccompanied by large direct stresses.*

299. The Occurrence of Shearing Stress in Practice.—Shearing stress is present in nearly all cases where there is cross-bending (see Art. 37), and in rivets, bolts, bridge-pins, crank-pins, etc., shearing stress becomes of practical interest. In none of these cases, however, is it found acting alone, but it is always combined with bending stress. In the case of rivets it is always combined with a very great tensile stress, caused by the contraction of the rivet in cooling after the heads have been made, this stress from contraction in good work always exceeding the tensile stress of the rivet at its elastic limit. In fact a riveted joint, if well made, always acts by frictional resistance alone, since this is always more than the working stress on the joint. (See a discussion of this subject in Chap. XXVI.) While rivets are computed for shear, therefore, as a matter of fact they are seldon subjected to a shearing stress.

* Both Dr. Kennedy and Mr. Barba grooved their specimens for double shear, and also held them in rigid forms. See *Rep. French Commission*, vol. III, Plate XIX.

For these reasons a knowledge of the true shearing strength of any of the metals is of little value, except for purely scientific purposes, and for computing resistance to torsion, where the stress developed is that of pure shear.

In the case of timber, however, which more often fails in shearing along the grain than in any other way, the strength in shearing is of great interest.

In general the shearing strength of the metals may be taken as 80 per cent of the tensile strength.

300. Shearing-test Appliances.—Shearing tests can be made in an ordinary tension or compression machine, if suitable appliances be used for holding the specimen. In Fig. 311 Dr. Kennedy's appliances for single and

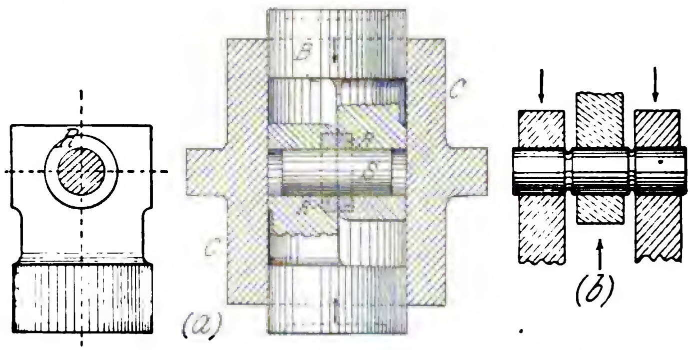


FIG. 311.—Dr. Kennedy's Appliances for Single and for Double Shear.

double shear are shown. For single shear the specimen is held by two half-rounds, all enclosed in a cylindrical sleeve. The shearing-faces are reinforced by steel rings. The plane of shear thus lies in the line of the axes of the two compression shear-blocks. For double shear the specimen is grooved on the shearing-planes, and it is also fitted closely into the eyes of the steel links through which the forces are applied.*

In Fig. 312 is shown the shearing-apparatus designed by the author for finding the shearing strength of cast iron. Here the specimen is gripped firmly at both ends and in the centre, and all bending distortion prevented. By preventing this kind of deformation the bending stresses are of necessity avoided. The bearing shear-plates at top and bottom are of hardened steel.

For shearing tests on wood the apparatus shown in Fig. 313 has been extensively employed by the author. Blocks about 2½ inches square and 8 inches long are slotted one inch from each end, in planes at right angles to each other, and also bored at the centre for the fixed hold. A rectangular steel pin is inserted in the slot, and the stick is prevented from splitting by attaching a clamp with an initial pressure just sufficient to hold it in place. The steel pin is pulled by means of bronze stirrups which are held in the

* This apparatus will not serve for cast iron. See *Trans. Inst. Civ. Engrs.*, vol. xc. p. 391.

regular wedge-grips of the testing-machine. After shearing out one end of the specimen it is turned over, the lower stirrup revolved 90°, and the other end pulled.

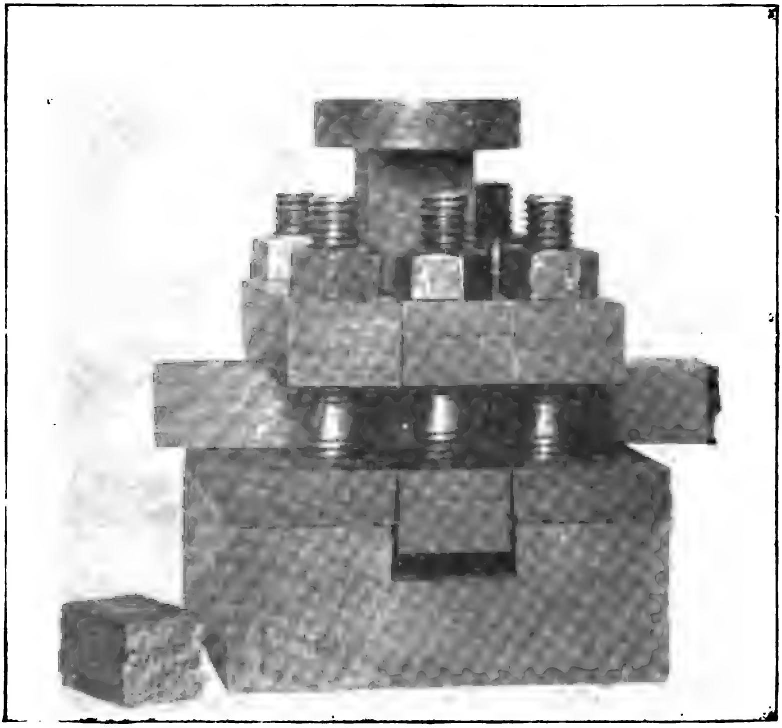


FIG. 312.—The Author's Shearing-test Apparatus.

TORSION TESTS.

301. Contrasted with Shearing Tests.—While torsional stress is a pure shearing stress (and about the only means of obtaining a pure shearing stress), yet a torsion test differs from a shearing test in that the deformation acts over any length of bar, taken at pleasure, and in that it is not uniformly distributed across the section, but is zero at the centre and increases uniformly towards the circumference. This enables the modulus of shearing elasticity to be determined by noting the angular distortion over a given length of bar, and it also makes possible the obtaining of autographic (or plotted) stress-diagrams for shearing stress. The elastic limit and ultimate strength in torsion have a value in the designing of shafting of all sorts which serve to transmit power.

302. Torsion-testing Machines.—In Fig. 314 is shown a simple attachment to an ordinary "universal" testing-machine. The power is applied to the specimen A by the screw-gear H, and the torsion is resisted by a couple

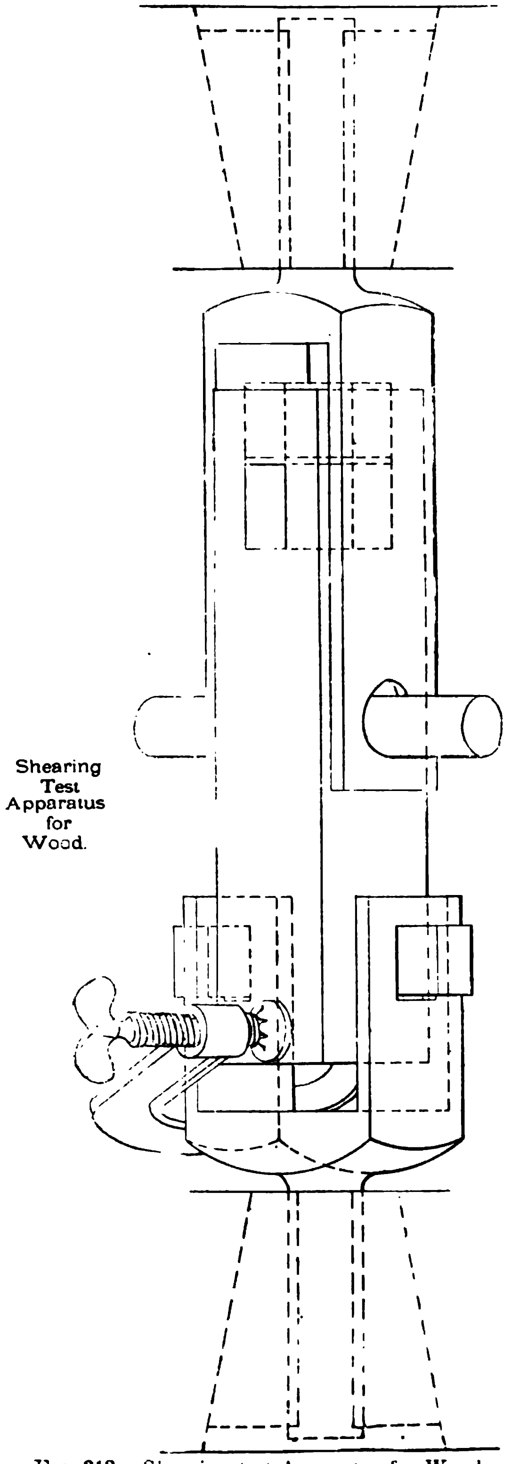


FIG. 313.—Shearing-test Apparatus for Wood.

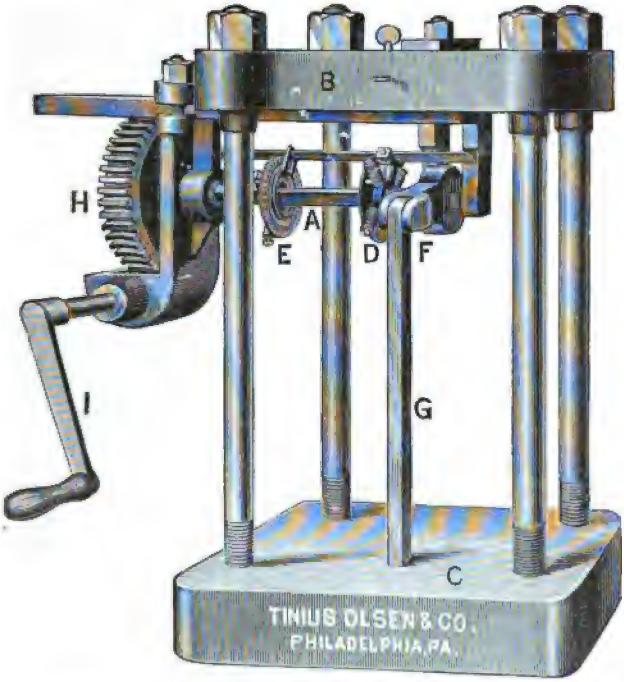


FIG. 314.—Torsion-test Attachment.

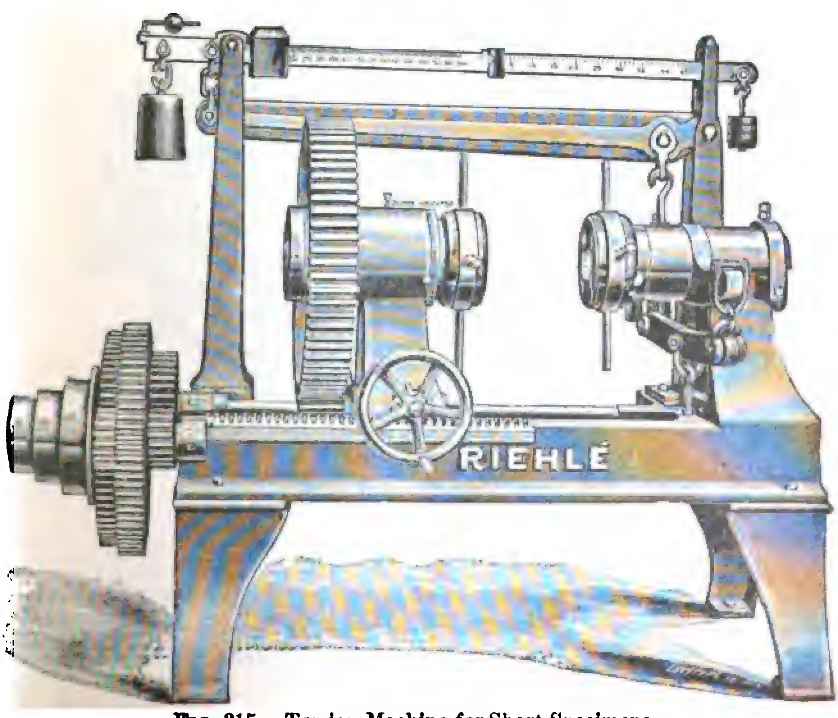


FIG. 315.—Torsion Machine for Short Specimens.

one arm of which, G, bears on the weighing-table. The angular deformation is observed by means of the two collars E and D, the latter holding rigidly a bar which moves a pointer over the graduated circle on the former.

FIG. 316.—Torsion Machine for Long Test-specimens.

In Fig. 315 is shown a torsion machine for testing large specimens in short lengths, while in Fig 316 is shown a large machine for bars of any desired length. The former machine is self-contained, while in the latter

FIG. 317.—A $3\frac{1}{4}$-in. square Bessemer-steel Bar Twisted Hot. (*Cassier's Mag.*, vol. x. p. 443, 1896.)

the lifting side of the weighing end is held down to the track by bolts, and the downward-bearing end of the couple-arm bears upon a system of weigh-

FIG. 318.—Tetmajer's Torsion-testing Machine for Wires, giving Autographic Records.

ing-levers. it is made in three sizes suited to steel shafts 1¼ in., 2 in., and 3½ in. in diameter, and for 16 feet in length or less. In this machine the specimen is free to contract longitudinally while under test.

FIG. 319.

A very perfect machine for testing wires from 0.05 in. to 0.18 in. in diameter (No. 18 to No. 7 B. W. G.) and for giving (*a*) the breaking moment, (*b*) the number of turns, and (*c*) the complete stress-diagram, is shown in Fig. 318.* This machine is used by Prof. Tetmajer and described by him

* Made by Messrs. Amsler-Laffon & Sons, Schaffhausen, Switzerland

in vol. IV. of his *Communications.* The specimen is kept in tension during the test by a weight suspended by a cord connected to the carriage at the resisting (and recording) end of the specimen. The resisting moment is developed by means of two weights suspended by cords which run in symmetrically arranged spiral grooves.

A simple machine without recording apparatus is shown in Fig. 319.

CHAPTER XX.

COLD-BENDING AND DRIFTING TESTS.

COLD-BENDING TESTS.

303. Their Character and Significance.—The test of the ductility of a malleable metal by bending it cold is the most common and perhaps the most useful of all the tests which can be applied to it. For wrought iron and structural steel this test approaches more nearly to the severe usages of actual practice than does the tension test with its elastic limit, ultimate strength, elongation, and reduction of area. It is not so easily standardized, however, and it is employed less in America than in Europe, partly because no standard methods and results have been agreed upon here. If a sample of wrought iron or steel will, when cold, fold upon itself absolutely, as shown in Figs.

FIG. 320.—Cold-bending Test of a 51-lb. 15-in. Steel Channel-bar. Thickness of web = 0.78 in. (*Engr. News*, vol. XXXIII. p. 272.)

320 and 322, or make the double fold as shown in Fig. 321, there can be no doubt of its high quality. When it fractures, however, at intermediate stages of this process, the question of its quality is left in doubt, and some standard limit is required if this test is to be made the basis of acceptance. The great advantage of this test is that it can be made at any time in the shop, without the expense attaching to tension tests, and by the man who uses or makes up the material. No standard method of making this test, therefore, should remove it beyond the range of ordinary shop appliances. In Europe a number of special machines are in use for making these tests,

but only shop-tools will here be assumed as available. With these methods, and in the hands of the same operator, uniform and comparable results may be obtained.

304. Methods of Making Cold-bending Tests.—If the specimen is not too large, a strong vise may be employed. If the bend is to be a true fold (radius of curvature = 0), the specimen should be bent about the sharp edge of the vise. If it is to be bent to a given radius, an auxiliary plate, dressed to this radius, must be clamped with the specimen in the vise. In either

FIG. 321.—Double Cold Bends on ⅜-in. Steel Plates. (*Eng. News*, vol. XXXIII. p. 272.)

case *the specimen must be clamped fast to a long steel bar, or lever*, so as to prevent all bending beyond the curved section. For this purpose two clamps * are required, one of which must be close down to the vise. The specimen is then bent to 90° by hand. Striking the specimen with a hammer should be avoided, as this kind of action cannot be standardized. If the specimen is to be folded flat upon itself, it may be removed from the vise after it has been bent to a right angle, and a second bar clamped to the other leg, and these two bars can now be drawn together by hand. The final closing down of the specimen may be done in a vise or under the hammer,—a steam-hammer always preferred.

The French Commission have adopted the *interior angle* as the index of the ductility. Thus if a straight bar bends through an angle of 60° before rupture, it leaves an angle of 120°, and this is the angle of record. A record of 0° signifies that the bar has bent through 180°, and that it has been either closed down flat or bent to a given radius, according as the radius of the bend is given as zero or something greater.

* Specially devised stirrups or clevises should be made up for clamping the specimens to the bending bar.

If the specimen is too large to bend by hand as described above, it may be bent under a steam-hammer (or in a hydraulic or screw press, or in a testing-machine, or even by a heavy sledge), by resting it on supports as a beam and striking it at the centre. After bending it in this way through an

FIG. 322.—Soft Bessemer-steel Bars, 3 in. by 2 in. in cross-section, Bent Cold. (*Cassier's Mag.*, vol. x. p. 442, 1896.)

angle of about 60°, it may be set on end and struck by the hammer (or placed in a press or testing-machine), as a bent column, and so brought down to any desired angle or radius of curvature. This required radius of curvature will have to be reached by flattening down the bent bar, after the zero angle

(180° of curvature) has been attained. The ideal appliance here is a press of some sort, but this requires a special machine (perhaps an ordinary "bull-dozer," used for straightening, or curving members, might serve), and in the absence of these a steam-hammer answers very well. A sledge is not good, as it is too light and requires blows having too high a velocity, which spend their energy in deforming the specimen at the point of impact and may produce its rupture earlier than the other methods would.

Prof. Tetmajer has a machine for making bending tests without the use of a mandrel, and whereby a uniform bending action is given to the bar. This develops the distributed elongation of the specimen, whereas a bend concentrated at one point develops the "reduction of area" quality. Thus a high-grade steel wire which will not elongate over two or three per cent may, on failure in tension, show a reduction of area of 60 per cent. Such a wire would fold over to a much sharper curve (smaller radius) than it could be bent to through a full circle.

Preparation of the Specimen.—If the specimen has been cut from a plate or from a structural form, and it is to be tested in comparison with or on the same basis as rolled bars, either round or rectangular, then the sheared edges should be removed by planing or filing where the bending is to be effected, in order to remove the brittle material resulting from the shearing action.

On the other hand, if it is desired to learn the action of the metal after it has been punched or sheared or threaded, then the specimen is purposely so prepared and tested without removing these hardened and serrated surfaces. The cold-bending test of such prepared specimens develops the injurious effects of these shop processes (punching, shearing, and threading) as nothing else can, and it is therefore necessary to use it for such purposes.

The French Commission have recommended a length of 10 inches and a width (of plate specimens) of 1.6 inches, the thickness to be that of the plate or bar.

Sometimes specimens are nicked or grooved on one side and then broken in cross-bending, under the hammer, to test relative brittleness. This is not a test that can be relied on to give absolute results, but Prof. Tetmajer used it to good effect to disprove the commonly accepted theory that even low steel is more brittle than wrought iron when subjected to shocks. Photographic views of the results of these tests are shown in Fig. 323. The depths of these specimens are shown in the figure. The six upper ones were 0.8 inch thick, and the four lower ones 1.2 inches thick. The tension tests on these specimens gave the following average results (eighteen tests on each material):

Material.	Modulus of Elasticity.	True Elastic Limit.	Apparent Elastic Limit.	Ultimate Strength.	Per cent of Elongation.	Reduction of Area
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	% in 8	%
Low steel......	31,000,000	28,500	36,600	61,000	27.8	59.3
Wrought iron..	28,600,000	21,800	33,000	52,200	16.0	21.4

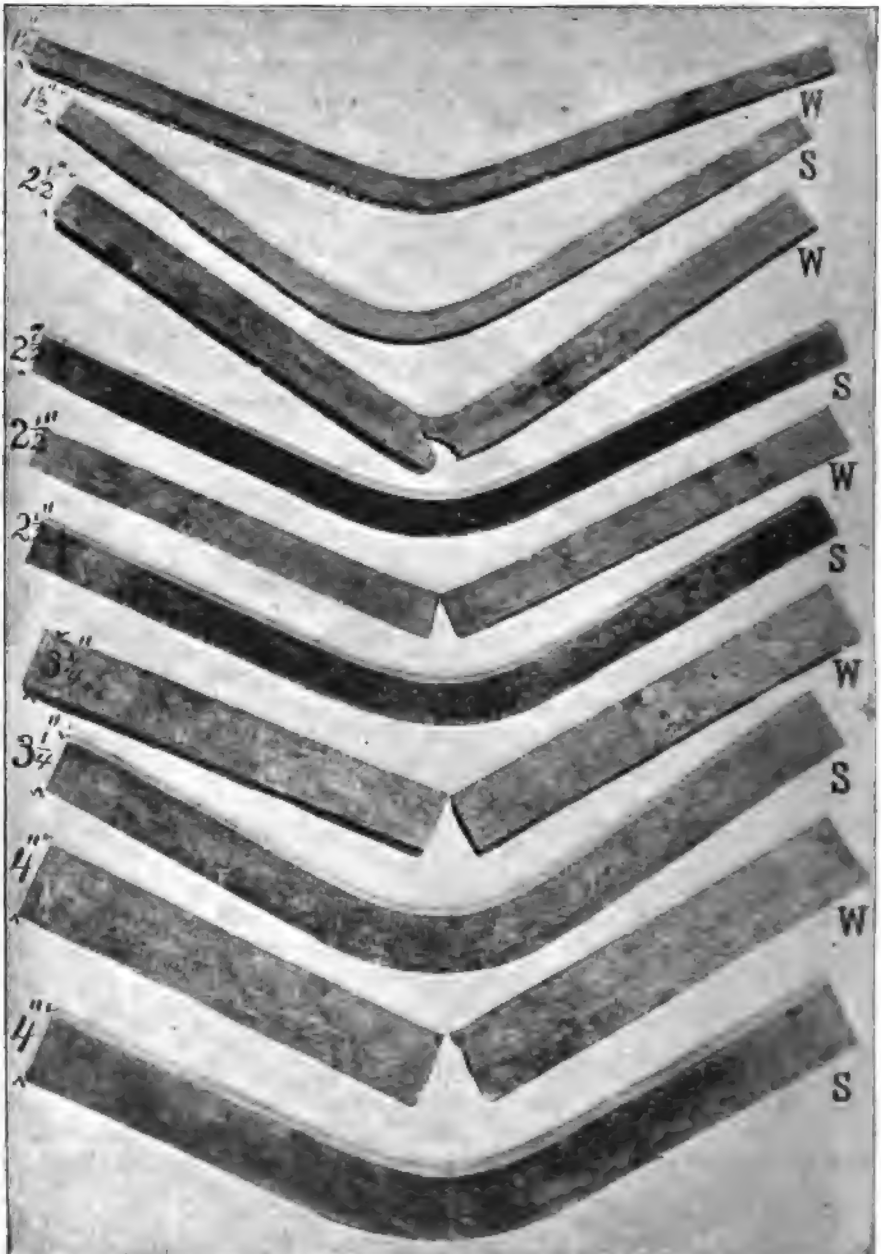
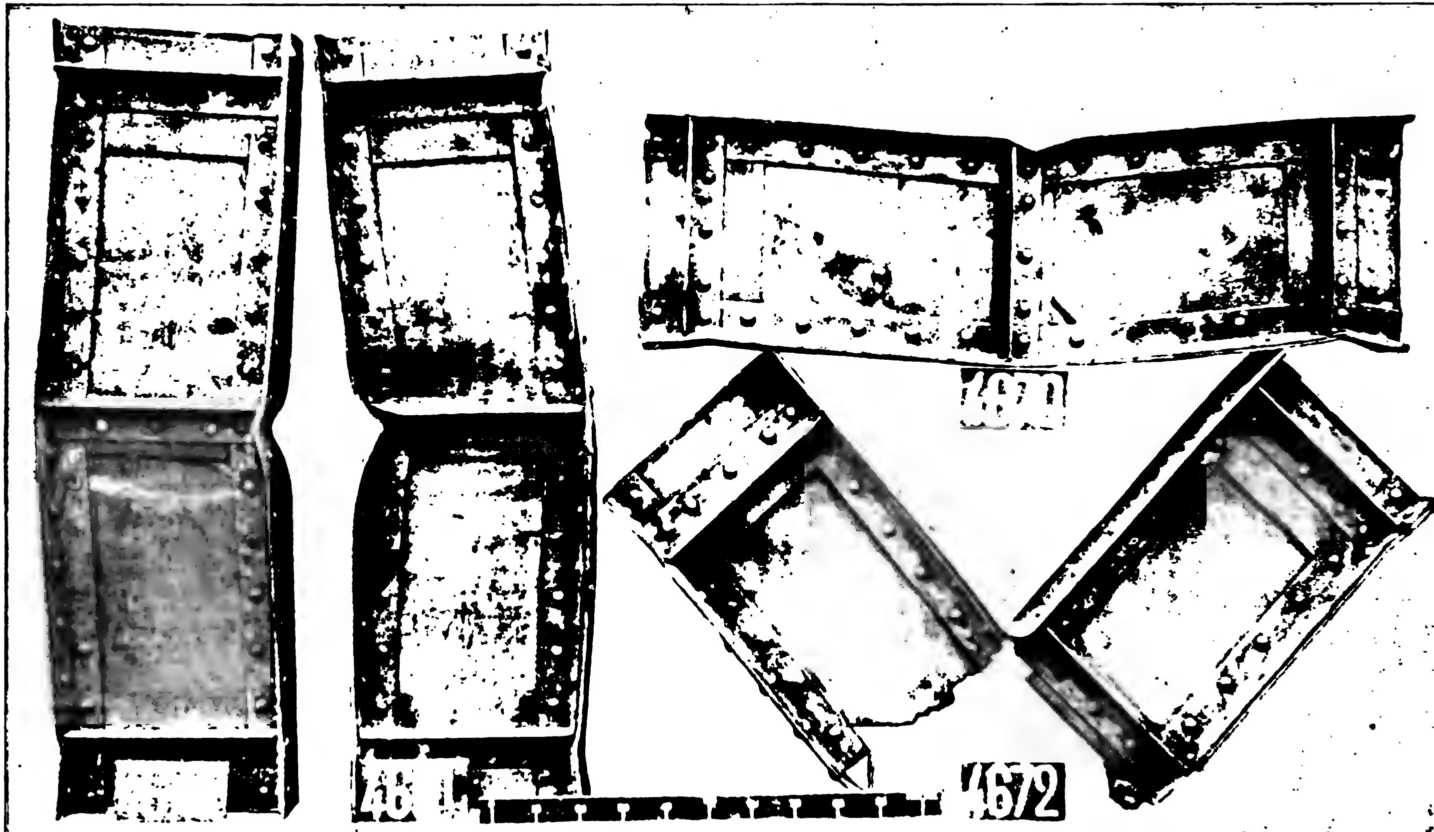


FIG. 323.—Showing Relative Resistance to Shock of Wrought-iron and Basic Bessemer Steel Bars which had been Grooved on the Tension Side to a depth of 0.06 in. (Tetmajer, vol. IV. Pl. X.) W = wrought iron, S = steel. The hammer weighed 660 lbs. and fell 9 in., 19 in., 29 in., 51 in., and 79 in. for the five sizes respectively, the span being always 40 in. The number of blows were 2, 3, 2, 2, 1, 4, 1, 7, 1, and 6 for the ten specimens as shown above, respectively.



This plate illustrates the relative resistance to impact of wrought iron and steel when made up into plate girders. These tests were made by Prof. Tetmajer, and the results are given in vol. IV. of his *Communications*. These girders were 20 in. high and rested on supports 6 ft. apart. Nos. 4670 and 4672 were of wrought iron, and Nos. 4671 and 4678 were of mild (60,000-lb.) steel. All of them were subjected to five blows of a ram weighing 2200 lbs., falling 4, 8, 12, 16, and 16 feet, respectively.

[To face p. 398.]

The great superiority of the steel in the impact tests is evident from the figure.

In the cold-bending tests on the same forms, flatwise, the steel specimens 0.8 inch thick folded flat without sign of rupture, while those 1.2 inches thick cracked in flattening down. The wrought-iron specimens 0.8 inch thick cracked after bending through an angle of 120°, and the 1.2-inch specimens after bending through an angle of 60°.

305. Comparison of Results of Cold-bending Tests with the Tensile Strength and the Percentage of Elongation.—In Baclé's report on Various Cold Tests of Materials, in the French Commission Report, vol. III. p. 311, are given the mean results of several thousand tests in tension and cold bending on wrought iron and soft steel, received from many different sources, made by M. Hallopeau, a member of that Commission, and given in the following tables. These results are not only of great value for the information itself, but they will serve to enable one to prepare standard specifications for cold-bending tests from the relation of the results of these to the results of tension tests which have long been the standard tests of acceptance in this country.

TABLE XVIII.—RELATIVE RESULTS OF TENSION AND COLD-BENDING TESTS.

Material.	Tension Test.			Bending Test.	
	Elastic Limit.	Ultimate Strength.	Elongation.	Cracked at Angle of	Ruptured at Angle of
	Lbs. per sq. in.	Lbs. per sq. in.	Percentage	Degrees.	Degrees.
Wrought iron, with grain......	32,000	48,500	14.0	80	30
" " across "	30,000	42,000	6.5	130	115
Low steel (57,000 to 60,000 T. S.)	29,500	60,000	27.0	0	0
Medium steel (64,000 T. S.).....	34,000	65,000	25.0	0	0
High steel (68,500 to 71,500 T. S.)	35,500	72,000	24.0	0	0

The tension-test bars were 4 in. long, 1.6 in. wide, and 0.4 in. thick.
The bending-test bars were 6 in. " 1.6 in. " " 0.4 in. "
The bending-test angle of record is the angle formed by the two ends of the bar after bending, or it is the supplement of the angle through which bending has taken place.

It will be seen from the above table that while the bending test would serve to distinguish varying qualities of wrought iron, it would not serve to distinguish these three grades of steel, since they all bent through 180° and folded flat without even cracking. Since these grades require distinction in practice because of the different degrees of injury produced on them by punching, the specimens can be punched and then bent, and the shades of hardness indicated by the greater angles at which cracks appear. This has been done by M. Hallopeau, and the results are shown in Table XIX. The plates were all the same size as those given in the previous table, the punched and drilled holes being 0.8 inch in diameter, or one half the width of the bar, the thickness being 0.4 inch. The die side of the plates was made the tension, or convex, side.

TABLE XIX.—EFFECTS OF PUNCHING AND DRILLING ON WROUGHT IRON AND STEEL, AS DETERMINED BY TENSION AND COLD-BENDING TESTS ON BARS 0.4 INCH THICK.

Material.	Tension Test. Elastic Limit.	Tension Test. Ultimate Strength.	Tension Test. Elongation.	Bending Test. Cracked at Angle of	Bending Test. Rupt'd at Angle of
	Lbs. per sq. in.	Lbs. per sq. in.	Percentage.	Degrees.	Degrees.
WROUGHT IRON.					
Plain.					
Natural { With the grain	32,000	48,500	14.0	80	30
Natural { Across the grain	30,000	42,000	6.5	130	115
Annealed { With the grain	20,000	47,000	19.8	65	20
Annealed { Across the grain	26,500	41,000	8.3	115	85
Hardened in water { With the grain	33,500	53,000	15.0	80	40
Hardened in water { Across the grain	39,000	46,000	8.3	135	100
Hardened and annealed { With the grain	28,500	47,000	16.7	100	50
Hardened and annealed { Across the grain	28,500	41,500	7.7	105	80
Punched.					
Natural { With the grain	29,000	38,500	1.0	175	150
Natural { Across the grain	26,000	32,000	1.0	180	175
Annealed { With the grain	42,000	46,000	2.4	170	150
Annealed { Across the grain	29,000	34,000	1.0	175	165
Hardened in water { With the grain	....	56,000	..	175	160
Hardened in water { Across the grain	33,500	38,000	1.0	178	175
Hardened and annealed { With the grain	....	43,500	..	170	150
Hardened and annealed { Across the grain	29,500	37,500	1.5	175	165
Drilled.					
Natural { With the grain	28,500	42,000	2.7	172	165
Natural { Across the grain	25,000	36,000	1.5	177	172
Annealed { With the grain	29,000	44,500	3.3	176	173
Annealed { Across the grain	24,500	33,000	1.8	177	173
Hardened in water { With the grain	36,500	54,000	1.8	175	168
Hardened in water { Across the grain	30,000	44,000	0.8	177	173
Hardened and annealed { With the grain	29,500	41,000	3.2	172	165
Hardened and annealed { Across the grain	27,000	35,000	1.5	178	175
LOW STEEL.					
Plain.					
Natural	29,500	60,500	27.0	..	..
Annealed	28,500	59,000	27.8	..	..
Hardened in water	34,000	65,500	22.4	..	..
Hardened and annealed	29,000	59,000	30.0	..	..
Punched.					
Natural	36,200	60,000	4.5	100	60
Annealed	36,500	55,000	4.6	90	59
Hardened in water	45,000	67,500	4.1	150	135
Hardened and annealed	36,400	58,000	5.0	90	60
Drilled.					
Natural	36,000	60,200	6.8	45	10
Annealed	34,200	59,700	7.4	40	5
Hardened in water	39,200	67,000	6 8	110	100
Hardened and annealed	33,500	59,700	7.1	20	10

EFFECTS OF PUNCHING AND DRILLING ON WROUGHT IRON AND STEEL—*continued.*

Material.	Tension Test.			Bending Test.	
	Elastic Limit.	Ultimate Strength.	Elongation.	Cracked at Angle of	Rupt'd at Angle of
MEDIUM STEEL.	Lbs. per sq. in.	Lbs. per sq. in.	Percentage.	Degrees.	Degrees.
Plain.					
Natural	34,000	65,500	25.0	..	..
Annealed	32,800	64,000	26.8	..	..
Hardened in water	37,000	73,500	23.8	..	..
Hardened and annealed	33,800	66,000	26.0	..	..
Punched.					
Natural	36,500	67,000	4.5	100	80
Annealed	35,500	64,000	4.7	95	75
Hardened in water	51,000	77,000	2.2	145	130
Hardened and annealed	37,000	64,500	4.7	100	75
Drilled.					
Natural	36,500	66,000	6.2	70	50
Annealed	35,500	64,500	6.5	65	40
Hardened in water	51,000	73,000	6.3	100	80
Hardened and annealed	36,000	65,000	6.5	60	40
HIGH STEEL.					
Plain.					
Natural	35,500	72,000	24.0	..	..
Annealed	36,200	70,000	25.0	..	..
Hardened in water	43,500	80,000	20.0	20 to 60	..
Hardened and annealed	35,500	71,300	25.0	..	..
Punched.					
Natural	47,000	72,000	3.9	140	105
Annealed	47,500	71,000	3.4	120	95
Hardened in water	71,000	80,000	3.4	160	145
Hardened and annealed	47,000	71.000	3.4	135	105
Drilled.					
Natural	39,700	71,800	5.5	80	60
Annealed	38,600	69,000	5.6	70	55
Hardened in water	56,700	82,000	5.8	140	125
Hardened and annealed	39,000	71,600	5.7	85	60

NOTE.—All the bars were 1.6 in. wide and 0.4 in. thick. The elongation in the tension test was measured on a length of 4 in. with both the plain and the punched or drilled specimens. In the former case this elongation occurred throughout this entire distance (4 in.), while in the latter it occurred only in the vicinity of the hole, but was credited to the entire distance of 4 inches in computing the percentage of elongation. These percentages are therefore not comparable as showing loss of ductility (as has been assumed in the Rep. French Com.).

The bending-test angle of record is the angle formed by the two ends of the bar after bending.

The following conclusions may be drawn from Table XIX:

1. The reduced ductility of wrought iron across the grain is fully brought out both in the elongation of the tension tests and in the angles of rupture in the cold-bending tests.

2. The weakening effect of both punching and drilling is very much greater with the wrought iron than with the soft and mild steels, and somewhat greater than it is on the medium steel.

3. The annealing of the punched specimens in no case appreciably increased their ductility, as shown by both the tension and the bending tests. It increased the strength of the wrought-iron specimens somewhat, but it lowered the strength of the steel specimens.

4. The drilled specimens of wrought iron do not differ appreciably from the punched either in strength or ductility, while with the steel of all grades the ductility of the drilled specimens is far greater than that of the punched specimens, although the ultimate strength is the same.

5. The change from 65,000- to 72,000-lb. steel is very clearly indicated by the bending test, where in the punched specimen, "natural," the angle through which the specimen bent before cracking is 100% greater with the former than with the latter. No such difference appears as between the 60,000-lb. and the 65,000-lb. steel, showing that they are about equally well adapted to such work. In the "plain" specimens all three grades of steel closed down entire (angle = 0) without sign of failure, thus manifesting no difference in hardness. The bending test on punched specimens, therefore, develops clearly this difference in fitness for riveted construction, and it might well be used as a shop-criterion of acceptance. Thus the 60,000-lb. and the 65,000-lb. steels bent through an angle of 80° after punching before a crack appeared, while the 72,000-lb. steel bent through an angle of only 40° before cracking. If an angle of 60° were specified on this test for plates 0.4 in. thick (leaving an angle of 120° formed by the two ends of the bar) before a crack should appear, it would seem to rule out the higher carbon-steels, which are injured by punching and shearing. This angle would be different, however, for different thicknesses of plate.

306. Combined Specified Requirements in Tension and Cold Bending.— The combined requirements given in Table XX are reproduced from the Report of the French Commission, vol. III. pp. 342–353. While the joint requirements of many French government bureaus are there given, only those of the *Artillerie de terre* are here given.

The Committee of the American Society of Civil Engineers has recommended (1896) the following cold-bending tests of plain specimens:

Wrought-iron specimens should bend through 90° without fracture, with inner radius not exceeding twice the thickness of the test specimen for bar-iron nor three times that thickness for plate and shape iron.

Rivet-iron and Rivet-steel bars, when heated to a low cherry-red and quenched in water (this for the steel bars only), must bend through 180° to a close contact (radius = 0) without sign of fracture.

Low Steel (60,000 lbs. T. S.), when treated in the same manner, must bend to a zero angle (through 180°), with an inner radius equal to the thickness of the specimen, without sign of fracture.

Medium Steel (65,000 lbs. T. S.) specimens, cut from bars, plates, or structural forms, in their natural state, must bend through 180°, with an inner radius equal to one and one-half times the thickness of the specimen, without sign of fracture.

High Steel (70,000 lbs. T. S.) specimens, cut from plates and forms, in their natural state, must bend through 180° to an inner radius equal to twice the thickness of the specimen without showing sign of fracture.

TABLE XX.—COMBINED REQUIREMENTS IN TENSION AND COLD BENDING.

Material.	Tension.		Thickness of Specimen. t	Cold Bending.	
	Ultimate Strength.	Elongation.		Angle before Cracking.	Radius of Bend.
WROUGHT IRON.					
Rolled Forms (Round and Rectangular).	Lbs. per sq. in.	Per cent.	In inches	Degrees.	
First-class charcoal iron, threaded[1]	48,500	25	$t < 1.6$	0[2]	$0.5t$
First-class puddled iron, threaded	48,500	25	$t < 1.6$	0[3]	$0.5t$
Good " " plain....	....	....	$t < 1.6$[4]	0	1.5 to $2.0t$
Common " " "	....	....	....	90	$2t$
Iron for bolts, threaded	48,500	25	$t < 0.6$	0	0 5t
" " " "	....	....	$t > 0.6$	90	$0.5t$
Plate Iron.[5]					
WITH THE GRAIN.					
First-class charcoal iron	....	....	$t < 0.4$	0	0
Refined puddled iron	50,000	10	$t < 0.4$	0	0 to $1.5t$
ACROSS THE GRAIN.					
First-class charcoal iron	....	....	$t < 0.2$	0	0
" " "	....	....	$t > 0.2$, < 0.4	90	0
Refined puddled iron	50,000	10	$t < .08$	0	0
" " "	....	....	$t > .08$, < 0.2	0	$1.5t$
" " "	....	....	$t > 0.2$, < 0.4	90	t
STRUCTURAL STEEL.					
Low and Medium.					
Rolled forms, hardened	48,000 to 60,000	26	any t	0	0
Plates,[5] "	48,000 to 64,000	23 to 25	any t	0	0
High Steel.					
Rolled forms, hardened	57,000 to 68,500	22	$t < 0.2$	0	0
" " "	....	....	$t > 0.2$	0	$\leqq t$
Plates,[5] "	60,000 to 71,000	21	$t < 0.2$	0	0
" "	57,000 to 68,500	21 to 23	$t > 0.2$	0	t

1. Screw-threads cut on bar where the bending occurs, as shown in Fig. 324.
2. This is the angle formed by the two ends of the bar after bending.
3. Some cracks are allowed here at the bottoms of the threads.
4. When the iron has a greater thickness than 1.6 in. it is to be cut down to this thickness.
5. Specimens sheared off and filed up smooth.

307. Comparison of Tension, Impact, and Cold-bending Tests.—It will be seen from the following tables that the loss of ductility in punching iron

and steel plates, and the benefit of subsequent annealing, are best developed by impact tests. Also, the benefits of enlarging punched holes by boring and reaming. The tables are compiled from M. Hallopeau's experiments described above, and are given in Rep. Fr. Com., pp. 356-7.

These test-bars were 8 in. long, 2.4 in. wide, and 0.32 in. thick. The punched and drilled holes were 0.8 in. in diameter, or one third the width of the plates. The hammer used in the impact test weighed 88 lbs., and it had a constant fall of 16 in. The average sums of all the heights of fall before cracks appeared are given in the table. The figures given are the average results of many tests.

TABLE XXI.—COMPARISON OF RESULTS BY TENSION, IMPACT, AND COLD-BENDING TESTS ON PUNCHED AND DRILLED PLATES.

Material.	Tension Tests on the Plain Specimens.			Impact Tests. Total Height of Drops.				Cold-bending Tests. Angles when Cracks appeared.			
					Holes Punched.				Holes Punched.		
						0.04 in. small.*				0.04 in. small.	
	Elastic Limit.	Ultimate Strength.	Elongation.	Holes Drilled.	Full Size.	Bored Out.	Reamed Out.	Holes Drilled.	Full Size.	Bored Out.	Reamed Out.
	lbs. sq. in.	lbs. sq. in.	%	in.	in.	in.	in.	deg.	deg.	deg.	deg.
Wrought iron, natural..	39,000	55,000	14.0	32	16	27	32	178	177	174	174
" " annealed	....	....	....	37	37	43	37	178	178	172	173
Steel, natural....	43,500	59,500	29.5	108	62	85	77	164	168	165	166
" annealed	....	....	....	130	107	128	112	159	161	160	161

* This is too small an enlargement to remove the material injured by punching, and hence these results do not fully develop the differences of treatment.—J. B. J.

TABLE XXII.—COMPARISON OF RESULTS OF THE IMPACT TESTS.

Relative Treatment of Specimens.	Wrought Iron.			Steel.		
	Annealed.	Not nnealed.	Excess of Annealed.	Annealed.	Not Annealed.	Excess of Annealed.
	inches.	inches.	%	inches.	inches.	%
Drilled full size..............................	37	32	14	130	108	20
Punched 0.04 in.* small and drilled out.....	43	27	59	128	85	50
Punched 0.04 in.* small and reamed out.....	37	32	14	112	77	45
Punched full size.........................	37	16	130	107	62	72
	%	%		%	%	
Superiority of drilling over punching	0	100	....	21	74	
Benefit of enlargement by drilling	16	69	....	20	37	
" " " " reaming..........	0	100	....	5	24	

* See note following Table XXI.

The following are some of the more important conclusions to be drawn from Tables XXI and XXII:

1. The great superiority under impact of the steel over the wrought-iron, with all kinds of treatment.

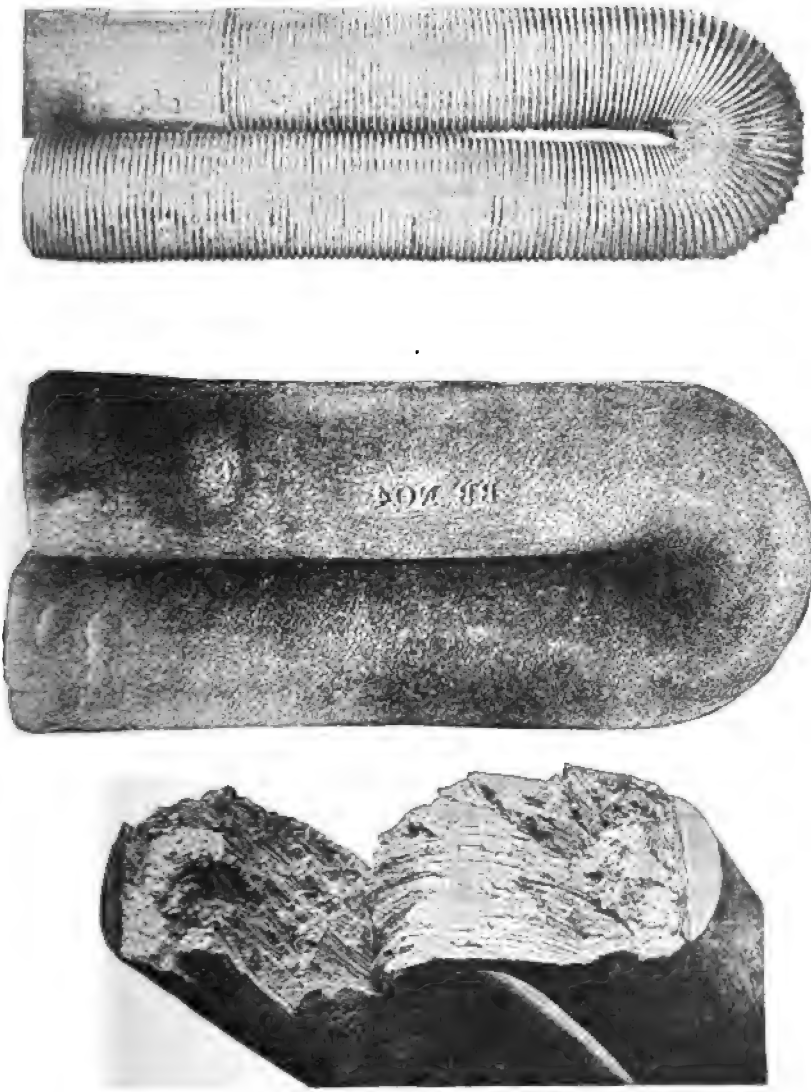


Fig. 324.—Cold-bending Tests of Best Wrought-iron, 1 in. to 2 in. in diameter. (From *Rep. U. S. Test Board*, 1881, vol. I.)

2. The excess in strength of the annealed over the unannealed specimens in all cases, with both iron and steel.

3. The superiority of drilling over punching in all cases, this being 100% with the wrought-iron and 74% with the steel plates, under the impact tests.

4. The great benefits of enlargement of punched holes by drilling or reaming, this being an average of 85% with the wrought-iron and 30% with the steel plates, when the thickness of plates was 0.32 in. and the enlargement only 0.04 in.

With greater thicknesses of plate the superiority of steel over iron would probably be somewhat less, while the differences indicated in 2, 3, and 4 would be greatly increased. With a greater enlargement of punched holes, also, the benefits of reaming would be much more marked, especially on the steel plates.

DRIFTING TESTS.

308. Their Character and Significance.—These, like the cold-bending tests, are such as may be applied in the workshop and by the workmen themselves with their ordinary shop appliances. The test consists in punching or boring holes of given diameters (varied with the thickness of the plate) at given distances from the edge of the plate or structural form, and then enlarging it by driving in it a drift-pin, as shown in Fig. 325, the percentage of enlargement without cracking being a very good indication of the ductility of the metal. To serve as a criterion of comparison, however, it must be reduced to fixed rules, the same as all other kinds of tests.

Fig. 325.—Drifting Test on $\frac{11}{16}$-in. Steel Angle. A $\frac{15}{16}$ in. Hole Drifted to $2\frac{5}{16}$ in. in Diameter. (*Engr. News*, vol. xxxiii. p. 272.)

A specification commonly used in France is as follows:*

Wrought-iron bars shall be cut both with and across the grain, 3 in. wide, and three holes punched, $\frac{5}{8}$ in. in diameter and $2\frac{3}{4}$ in. apart, along the central line of the plate. These holes shall then be enlarged, beginning with the central one, and using a drift-pin which increases its diameter at the rate of 1 in 10. Plates 0.20 in. thick should submit to an enlargement of the $\frac{5}{8}$-in. hole to a diameter of 1 in.; plates 0.25 in. thick should enlarge to 1.2 in. diameter; plates 0.30 in. thick should enlarge to 1.32 in. diameter; and plates thicker than 0.32 in. should enlarge to from 1 in. to 1.3 in., according to quality, without showing any sign of failure.

Steel plates, similarly prepared, of 57,000 lbs. tensile strength should enlarge to 1.6 in. diameter after annealing and to 1.5 in. diameter after hardening in water. Steel plates of 57,000 to 64,000 lbs. tensile strength should allow a $\frac{5}{8}$ in. hole to enlarge to 1.5 in. diameter after annealing and to 1.4 in. diameter after hardening in water.

* That of the Eastern Railway Company.

CHAPTER XXI.

THE TESTING OF CEMENT.

309. The Standard Scientific Tests of Cement are those which are made to determine the following properties:

(*a*) Strength, neat and with different proportions of sand;
(*b*) Fineness of grinding;
(*c*) The thoroughness of the burning;
(*d*) The rate of setting;
(*e*) The permanency of volume, commonly called the test for "soundness."

The strength of cement and of cement-mortar is usually determined by the tensile test on small shapes, called briquettes, which have hardened under water for varying periods of time. The more common periods are: for natural cement, one day and seven days; for Portland cement, seven days and twenty-eight days. It is well, however, to extend the time of setting to a longer period if practicable. Since natural cement usually sets and hardens more rapidly than Portland cement, it is sometimes used in place of the Portland, where but a short period of time can be allowed for the testing. Thus for street improvements the material is usually tested after it is brought upon the works, that is to say, placed upon the sidewalks; and as it here forms a serious obstruction, it is desirable to have the tests made in as short a time as possible. Since the one-day test for a quick-setting natural cement will indicate its quality, such a material is often used solely on this account.

Although cement is more commonly subjected to compression, yet it has been found that the tensile test effectually indicates the compressive strength (see Fig. 337). This holds true both for the neat cement and for cement-mortars.

Since cement is always used mixed with sand, some of the highest authorities are now advocating the abandonment of the neat-cement tests for strength and making the strength test on a mortar containing three of sand to one of cement, by weight, in the case of Portland cement, and two of sand to one of natural cement, by weight, these being the usual proportions. For special purposes four or five parts of sand may also be employed,

especially with finely-ground cements, or such as give a residue of less than 10 per cent on a sieve having 14,400 meshes per square inch (2300 per square centimeter). Since in the sand mixtures a standard sand must be employed, it has become customary to use clean, sharps and which has passed a No. 20 sieve (20 meshes per linear inch), and stopped on a No. 30 sieve (30 meshes per linear inch). In order to further insure identity of the sand used, the American Society of Civil Engineers, has recommended that crushed quartz be used, such as is employed in the making of sandpaper. The author does not favor this practice. This material has fully 50 per cent of voids, while the ordinary sands, with roughly rounded grains, contain but about 33 per cent of voids. Any good, sharp, clean sand, therefore, of the size 20–30 should give very nearly uniform results which will average much higher than those obtained with crushed quartz, unless the quartz briquettes be thoroughly compacted by hard hammering.

All tensile-test briquettes of Portland cement (neat or with sand) should be kept in a moist atmosphere for 24 hours, and then kept the remainder of the period under water. Natural cements are kept from one to four hours in air (or till they have set) and then put in water.

The importance of maintaining the water for mixing, and for the bath during the entire hardening period, at a standard temperature, in order to

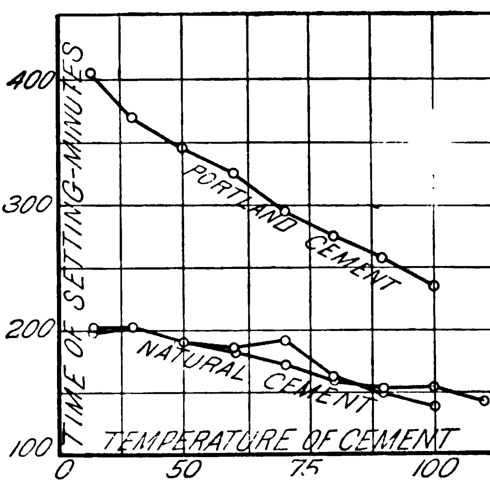


FIG. 326.—Effect of Temperature of Cement on Time of Setting. (Wheeler, *Rep. Chf. Engrs.*, 1895, p. 2936.)

obtain uniform results, is clearly shown by Figs. 326 and 327. In the former it is shown that the time of setting is greatly shortened by increasing the temperature of the mixing water, while Fig. 327 indicates that the strength attained in a given time may be greatly increased by raising the temperature of the bath from 40° to 80° F. In the case of normal mortar, 1C. : 3S., this increase, at two months, was from 100 lbs. to 230 lbs. per square inch. The Fifth International Convention for Unifying the Methods

of Testing Materials (Zurich, Sept. 1895) decided that it was not advisable to hasten the hardening process by raising the temperature of the bath, since, after numerous trials, uniform results could not be obtained.

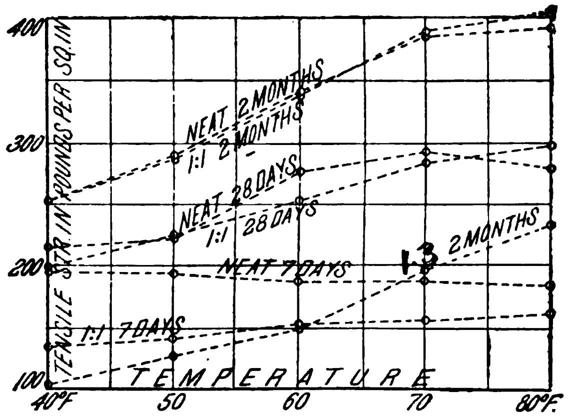


FIG. 327.—Showing Effect of Temperature of Immersing Tanks on the Rate of Hardening of Natural Cement-mortar. (Wheeler, *Rep. Chf. Engrs.*, 1894.)

310. The Fineness of the Grinding is determined by passing the cement through sieves of a specified number of meshes to the lineal inch. While

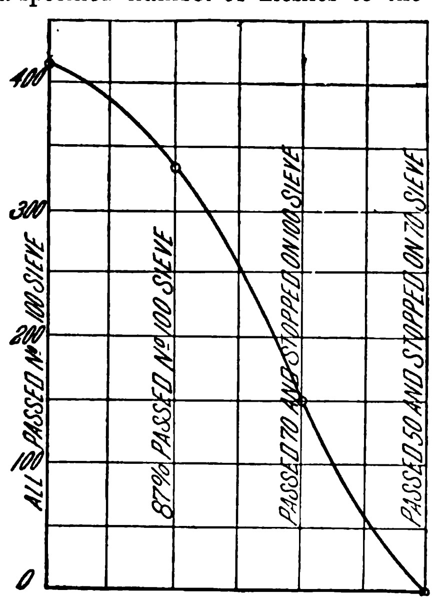


FIG. 328.—Showing Absence of Cementing Properties of the Coarser Particles of Cement, 1 C. : 3 S., age 4 mos. (*Jour. Assoc. Eng. Socs.*, vol. XIV. p. 245.)

the size of such meshes would of course depend on the diameter of the wire used, it is difficult to determine this diameter, while the counting of

the meshes is practicable. It has been found by experiment, Fig. 328, that only the finest or most impalpable dust is really active in the setting and hardening of the cement, the coarser grains acting as so much inert matter, which might as well be replaced by sand. The proportion of the cement which passes a sieve of less than about 100 meshes to the lineal inch does not give any intelligent idea of the significant fineness of the grinding. In fact the standard sieve for determining fineness now generally used on the continent of Europe has seventy meshes per lineal centimeter, which corresponds to 175 meshes per lineal inch, or over 30,000 meshes per square inch. Not more than about twenty-five per cent of the cement should be held on a sieve of this degree of fineness.* The author of this work recommends that a sieve of 120 meshes per linear inch (14,400 per square inch) be used, and that the residue on this sieve shall not be more than twenty (20) per cent. This requirement can now be readily complied with by all the leading manufacturers of Portland and slag cements.†

The French Commission advocate, in testing for fineness—

1. Separating it into four grades by using sieves as follows:

Approximate Number of Sieve.	Number of Openings per Linear		Number of Openings per Square		Size of Wire		Size of Openings	
	Inch.	Centimeter.	Inch.	Centimeter.	In Inches.	In Millimeters.	In Inches.	In Millimeters.
50	50	18	2,500	324	0.08	0.20	0.014	0.36
80	80	30	6,400	900	0.06	0.15	0.007	0.18
175	175	70	32,400	4,900	0.02	0.05	0.0035	0.09

2. This test to be made on a sample of 100 grams, with sieves about 12 inches in diameter.

3. Hand-sifting to be considered finished when not over 0.1 gram passes under the action of 25 movements.

4. The employment of a shaking-machine is recommended, especially for the No. 175 sieve.

5. The results should be given as the total percentages which failed to pass each sieve, beginning with the finest. Thus the percentage held by the No. 175 sieve includes the percentages stopped on the other two, and the percentage given for the No. 80 sieve would include that held on the No. 50 sieve.

* It has been customary in the United States to specify a sieve of 50 meshes per lineal inch, but occasionally a sieve of 100 meshes per inch has been used. The former size has no significance whatever in determining that degree of fineness requisite to proper action of the cement, and the latter is too coarse to have much or any value.

† This is also the standard chosen by Mr. J. W. Sandeman, M. Inst. C. E., in Trans. Inst. C. E., vol. CXXI. (1894–5) p. 215, and it has also been adopted by some officers of the U. S. Engr. Corps.

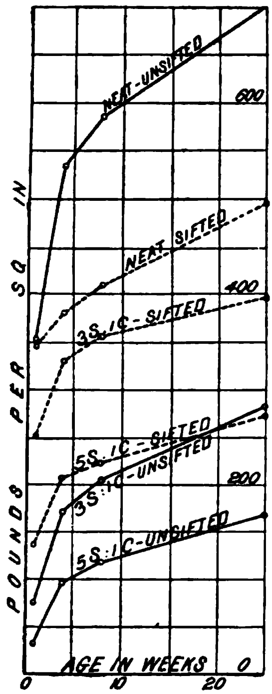


Fig. 329.—Showing Effect of Sifting a Coarse Portland Cement through a No. 180 mesh sieve (*Trans. Inst. C.E.*, vol. 84.)

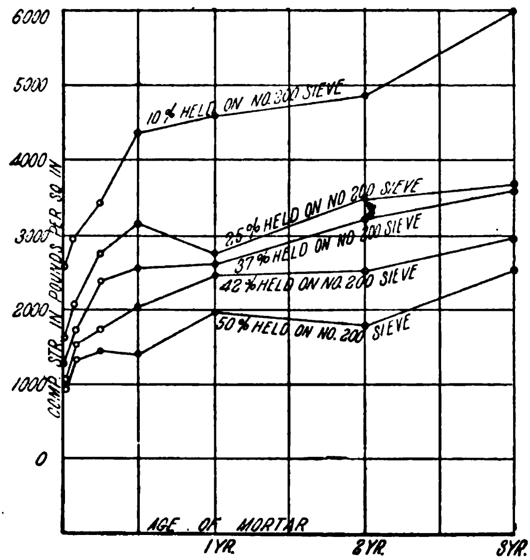


Fig. 330.—Showing the Greater Efficiency of Very Finely Ground Cements when mixed with Three Parts of Sand. (Tetmajer.)

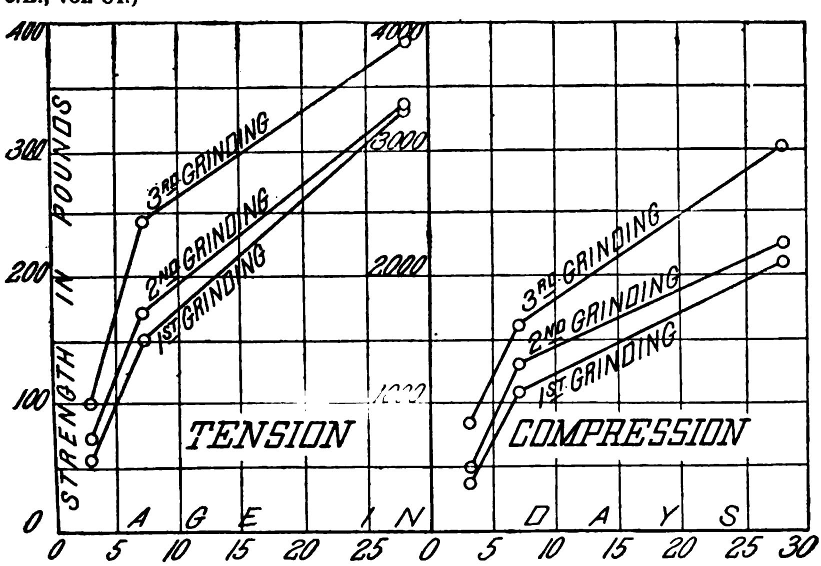


Fig. 331.—Showing Effect of Regrinding Portland Cement on Mortar, 1 C. : 3 S. First grinding left only 6.5% on a No. 175 sieve (30,000 meshes per square inch). After second grinding it all passed this sieve. (Tetmajer, vol. VII.)

Dr. W. Michaelis, the great German specialist, recommends* that two sieves be used, No. 75, and No. 150 (30 and 60 meshes per cm.), and in addition to these the Schöne washing apparatus with rates of upward flow of the alcohol of 2.8 inches per minute, giving particles of cement which would pass a No. 300 sieve (120 per centimeter), and also of 1 inch per minute upward velocity, giving particles which would correspond to those passing a No. 600 sieve (240 meshes per centimeter). This washing process, added to the use of the two sieves, would enable one to graduate the cement as follows:

Number of Meshes per		Diameter of Wire		Width of Mesh		Area of Mesh	
Square Centimeter.	Square Inch.	In Millimeters.	In Inches.	In Millimeters.	In Inches.	In Square Millimeters.	In Square Inches
900	4,200	0.133	0.0052	0.20	0.0080	0.04	0.0000610
3,600	23,500	0.067	0.0026	0.10	0.0040	0.01	0.0000150
15,000	97,000	0.033	0.0013	0.05	0.0020	0.0025	0.0000040
60,000	390,000	0.002	0.00008	0.02	0.0008	0.0004	0.0000006

The relation between the largest diameter of particle and the rate of upward flow for absolute alcohol and Portland cement he finds to be

$$d = 0.036v^{\frac{7}{11}},$$

where d = largest diameter in millimeters, and v = upward velocity of flow in millimeters per second in the cylindrical part of the washing apparatus.

As a result of this further analysis for fineness it appears that the conclusions drawn from an analysis with the No. 75 and the No. 175 sieve (30 and 70 per centimeter) may be entirely erroneous. Thus among the many analyses given by Michaelis in these articles are the following two analyses of cement ground in the same manner, on French buhrstones, 5 ft. in diameter:

Sieve-gauges (Meshes per Linear Inch), where Diameter of Wire = Width of Mesh.	Sample No. 1.		Sample No. 2.	
	Parts.	Total Passing.	Parts.	Total Passing
Retained on No. 75 sieve	0.65%	99.35%	1.55%	98.45%
Passed No. 75 and retained on No. 175 sieve	7.75	91.60	7.40	91.05
" " 175 " " " " 300 "	42.98	48 62	19.74	71.31
" " 300 " " " " 600 "	17.75	30.87	25.27	46.04
" " 600 sieve	30 87		46.04	
	100.00		100.00	

* *Thonindustrie-Zeitung* (*Clay-industry Gazette*), Berlin, Aug. 24 and Nov. 23, 1895. Dr. Michaelis first introduced the No. 175 sieve in Germany about 1875.

The total percentage passing the No. 175 sieve was 91.60 for sample No. 1, and 91.05 for sample No. 2. This would appear to give No. 1 a slight advantage. There was stopped at the next stage, however, 43 per cent of No. 1 and only 20 per cent of No. 2, thus leaving only 48.62 per cent of No. 1 to pass the 300 sieve, while of No. 2 there passed 71.31 per cent. Finally, there was but 31 per cent of No. 1 to pass the washing test which corresponded to a No. 600 sieve, while 46 per cent of No. 2 passed this last test of fineness. It thus appears that sample No. 2 is much finer ground than No. 1, although this would not appear from the most severe sieve-test it is possible to make, it being impracticable to use any finer sieve than about 175 meshes per linear inch (70 per centimeter).

Mr. Michaelis strongly urges, therefore, that in all scientific and expert investigations of fineness the washing test be employed.

311. The Thoroughness of the Burning is indicated by the specific gravity of the ground cement. If the cement is underburned, it is relatively light. This test is, therefore, really a test for specific gravity.

Since the volume of a given weight of cement depends altogether on the way in which it is shaken down or compacted, it is impracticable to determine specific gravity by weighing measured volumes. The specific gravity of cement is found, therefore, by means of an apparatus like that shown in Fig. 332. This vessel is filled with benzine or turpentine,* up to the zero graduation on the inserted tube or above. A definite weight of cement is slowly dropped into the top of this tube, care being taken to allow all air-bubbles to escape, when the rise of the liquid in the tube will indicate the true volume of the cement which has been added. If metric units have been used, then the specific gravity of the cement is equal to the weight of the quantity added in grams divided by the increase of volume in cubic centimeters. Since well-burned Portland cement has a spceific gravity of more than

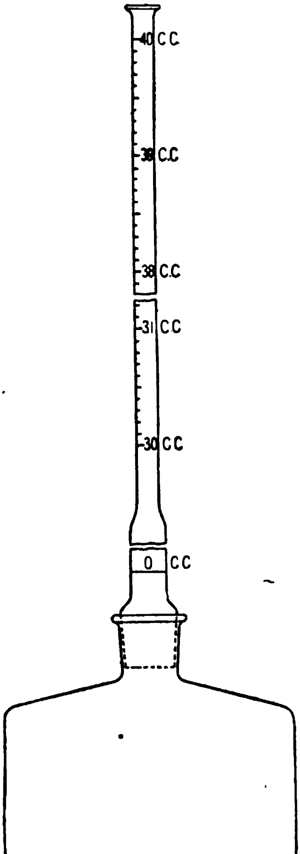


FIG. 332.—Apparatus for Determining the Specific Gravity of Cement, as used by the Author. One-third natural size.

* While the cement has no tendency to set or harden when turpentine is used, yet the volume of this liquid is so sensitive to changes in temperature that it is not advisable to use it unless the cement, the turpentine, and the vessel all have the temperature of the room, and this latter remains constant during the test.

3.05, this figure may be taken as a minimum specific gravity to be used in a specification.

In making this test it is necessary to see that all lumps are thoroughly

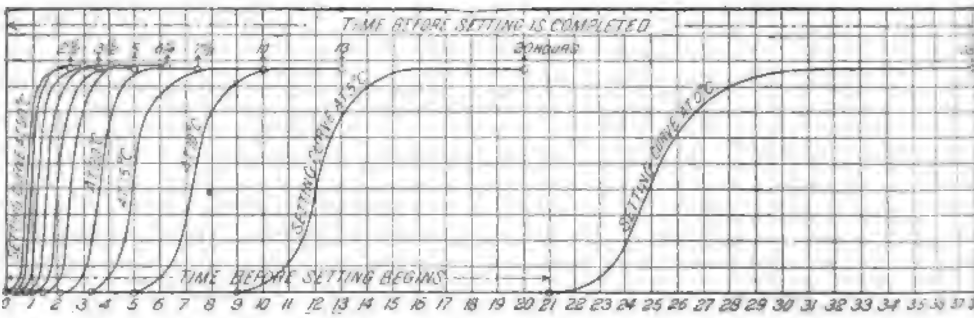


FIG. 333.—Graphical Representation of the Rate of Setting of Portland Cement at Various Temperatures, automatically recorded by the Apparatus shown in Fig. 334. (Tetmajer.)

pulverized and dried. To insure this it should be passed through about a No. 80 sieve, that remaining on the sieve to be added to the specimen. This

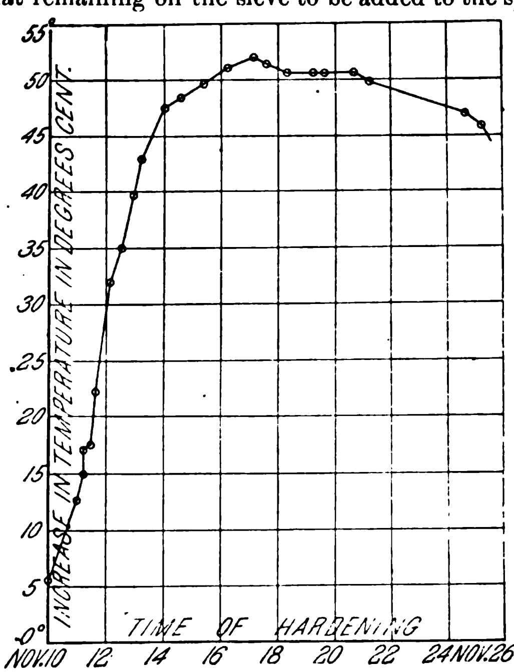


FIG. 333a.—Increase in Temperature at the Centre of a Heavy Mass of Portland-cement Concrete 11.5 ft. thick, as a result of Chemical Action. (*Rep. Fr. Com.*, vol. IV, Pl. III.)

test should be accurate within one per cent, or to within three in the second decimal place.

This test is not usually applied to natural cements, since it is not supposed that they will be burned either so carefully or at so high a temperature as is required for Portland cement.

312. The Rate of Setting.—In the process of hardening of cement-mortar there are two well-defined stages, known respectively as the beginning and the ending of the setting. A quick-setting cement may begin to set within a very few minutes after wetting, while a slow-setting cement may require

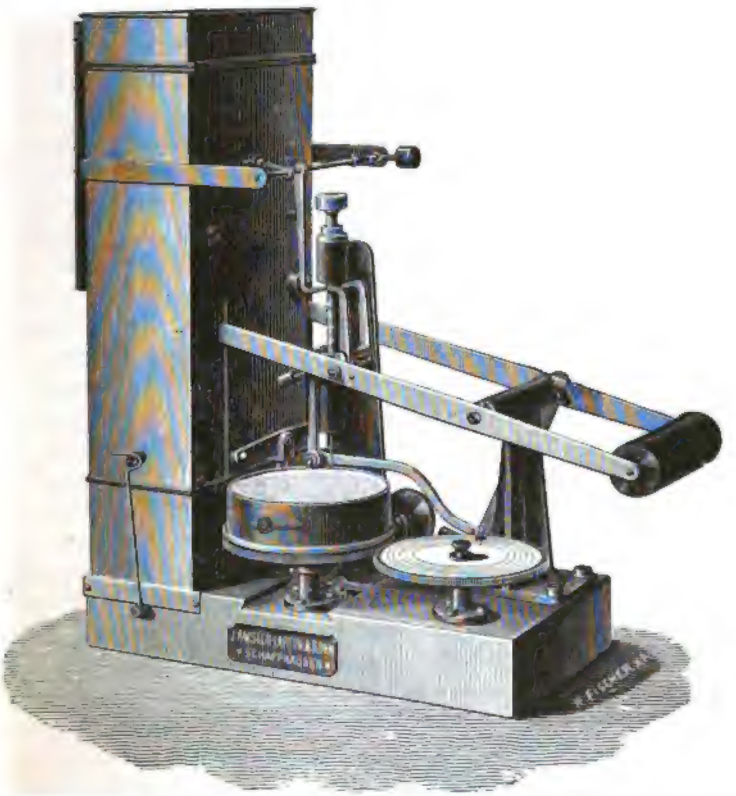


Fig. 334.—The Amsler-Laffon Apparatus for Automatically Registering the Rate of Setting of Cements as given in Fig. 333.

more than twenty-four hours before it begins to set. Usually the setting progresses rapidly after it has begun, as indicated in the curves in Fig. 333. These curves have been automatically recorded * by the apparatus shown in Fig. 334. This setting action is always accompanied by a slight rise in temperature. In fact, with quick-setting cements the temperature curve is a truer index of the setting period than the mechanical tests of firmness which are usually employed for this purpose. As long as the temperature

* Taken from Prof. Tetmajer's Reports, vol. VI.

continues to rise the setting action is in progress, and the rate of setting is well indicated by the rate of increase of temperature.

An excellent illustration of the evolution of heat by chemical action in the hardening of cement is furnished by Fig. 333*a*. Here the rise in temperature was observed for sixteen days at the centre of a mass of Portland-cement concrete 11.5 feet thick. The temperature rose 47.5° C. (85.5° F.) in four days, and reached its maximum increase of 52° C. in seven days, after which it fell off very slowly. Both the temperature-curve and the hardening-curve are automatically recorded by the apparatus shown in Fig. 334. This apparatus was manufactured by Messrs. J. Amsler-Laffon & Son, Schaffhausen, Switzerland. Its operation is too complex to be explained here. It is in satisfactory use, however, in Prof. Tetmajer's laboratory at Zurich. With slow-setting cements the rise of temperature cannot be observed with accuracy, and is smaller in amount than in the case of quick-setting cements. In such cases the heat developed is dissipated because of its slow generation, and does, therefore, not become sensible to thermometric measurement.

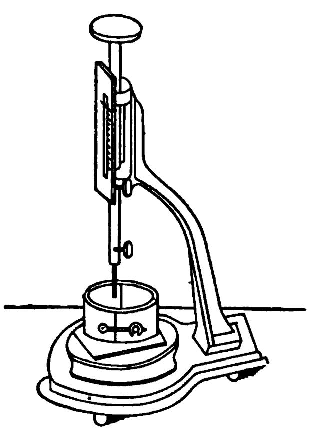


FIG. 335. — The Vicat-needle Apparatus.

The usual method of determining the setting period is by means of such an apparatus as shown in Fig. 335. By this means a needle of a particular diameter and loaded with a specified weight is allowed to rest upon the cake of mortar, which for this test should be mixed neat, and the setting is determined by the depth of penetration of the needle. When the needle ceases to reach the bottom of the cake, setting is supposed to have begun; and when it rests wholly on the top, the setting is supposed to have been completed. The temperature-curve, however, indicates a continuation of this action for some time after the needle ceases to penetrate the mass. The method commonly employed in America is that recommended by the American Society of Civil Engineers, which is as follows:

A neat cement-mortar having a stiff, plastic consistency is placed in a form two or three inches in diameter and one-half inch thick. When a needle one-twelfth inch in diameter, weighted with one-fourth pound, ceases to penetrate the entire mass, setting is said to have begun; when a needle one twenty-fourth inch in diameter, carrying one pound, will not penetrate the mass at all, setting is said to have been completed.*

In this test, those cements which set completely in one-half hour or less are known as quick-setting. Those requiring much more time, slow-setting. It must not be supposed that these terms are used rigidly with this limit.

* The French Commission recommend a needle 1.13 mm. diameter and loaded with 300 grams for both of these tests. This needle is 0.045 inch diameter and just 1 square millimeter in area. The load is 11 ounces.

In Germany and France a needle one millimeter in diameter is loaded with a weight of 300 grams, and the beginning and the end of the setting period is indicated by the time when this needle ceases to penetrate the entire mass, and when it ceases to penetrate it at all, respectively.

The time when the setting has been completed can be approximately determined by efforts to indent the surface with the finger-nail. When the surface offers some appreciable resistance to such indentation, the cement may be said to have set. From results of tests given in Chapter XXX it does not seem to be as injurious to the final strength of the mortar to use it after it has begun to set as it has commonly been supposed.

313. The Test for Soundness.—This is a test of the permanency of volume of a cement-mortar, or of its resistance to disintegrating influences. Although wrong mixtures, improper calcining, and coarse grinding may lower the strength of a cement, a strong tendency to swell or to disintegrate is absolutely fatal, not only to the mortar, but also to the structure in which it is used. In America reliance in this matter has been placed on the good record of the particular brand, rather than on actual tests to determine the "soundness." Professor Tetmajer of Zurich, the leading authority now on cement-testing, having tried various methods of determining this property, recommends for Portland cement the boiling test described below. For slag-cement and for the natural cements he has not found any satisfactory means of determining this quality by a short test.

While "soundness" may be tested by a long maintenance of the cement-cakes under water and in the air for many years at least, this is of course not possible in practice.

It is true that all cements swell under water and shrink in the air, but these changes are usually inappreciable. A dangerous swelling of volume under water may be caused by an excess of quicklime (CaO) in an overburnt condition, which resists the slacking action of water for a considerable time. These cements may be called "lime-expanders."

The sulphur compounds of lime (CaS and $CaSO_4$, or gypsum) may cause the cement to disintegrate in air by oxidation and the absorption of water (becoming $CaSO_4 + 2HO_2$ and $CaSO_4 + 7H_2O$).

If magnesian limestone, or dolomite, forms a considerable portion of the raw ingredients, after years of apparent soundness, the cement may disintegrate from swelling, if under water, due to the final slacking of the magnesia. Professor Tetmajer states, however, that he has never met with any actual Portland cement which has failed in a test from the presence of the sulphur or the magnesia compounds. It seems, therefore, that the only source of unsoundness to be anticipated is an excess of quicklime, and this is best determined by the boiling test.

314. The Boiling Test.—This test has been practised for the past twenty-five years, and has received almost universal sanction. At the Fifth International Convention for Unifying Methods for Testing Construction

Materials, held in Zurich, Sept. 1895, the following rules for conducting this test were recommended by a committee of the leading experts of Europe, Dr. Michaelis being chairman, who originally proposed this test about 1870.

I. The rapid test of hydraulic cements for constancy of volume consists in the application of warm baths at temperatures of from 50° to 100° C. (122° to 212° F.).

II. *Manner of Making the Test-pieces.*—Enough water is used to bring the neat cement, after proper working, into a plastic state. Two balls from 40 to 50 millimeters (1.5 to 2 inches) in diameter are formed by hand and kept in moist air, resting on some nonabsorbent material. (Sand mixtures are not to be subjected to this test, neither are briquettes which are to be tested for strength to be so treated.)

The employment of tension briquettes and cylindrical disks from 50 to 100 millimeters (2 to 4 inches) in diameter and from 15 to 30 millimeters (¾ to 1¼ inches) in thickness is likewise permitted.

III. *Duration of Previous Hardening.*—Until set has taken place test-pieces must be kept in moist air. Portland, slag, Pozzuolana, and Roman cements will be uniformly kept thus for twenty-four hours; very slow-setting ones for forty-eight hours. Hydraulic limes and all cements that have not completely set after forty-eight hours will be allowed seventy-two hours for previous hardening.

IV. *Treatment in the Warm Bath.*—The previously hardened test-samples are placed in a water-bath at ordinary temperature, which is then gradually—not in less than thirty minutes—heated to the prescribed temperature and kept there. After three hours at the prescribed temperature the test is interrupted, the test-pieces are taken out of the bath, and after having cooled sufficiently, examined as to their condition. They must not be chilled suddenly by means of cold water.

For each warm-bath test the water must be renewed. The temperature of the bath will be:

For Roman cements and hydraulic limes, 50° C. (122° F.); for Portland slag and Pozzuolana cements, 100° C. (212° F.).

V. In order to be considered of absolutely constant volume the test-sample must, during this test, remain perfectly sound and entirely free from cracks and warping.

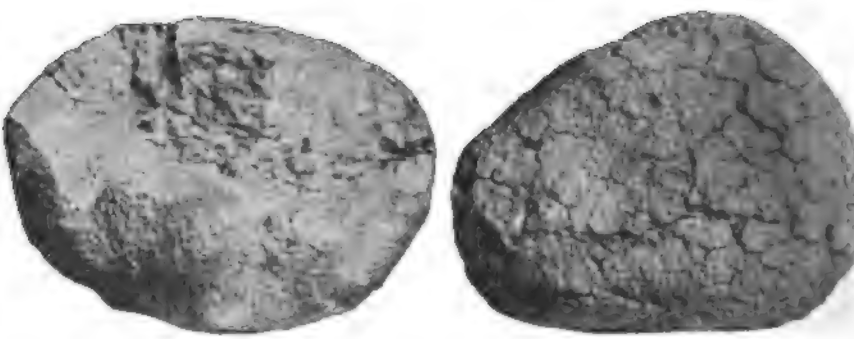


FIG. 336.—Showing Methods of Failure of Cements under the Boiling Test.*

If the ball cracks slightly in this test or disintegrates somewhat as shown in Fig. 336, it should be considered at least doubtful, although it might not fail in actual practice.

A modification of this test is to maintain a bath at a little less than the boiling temperature, in order to prevent the wearing action of the boiling water. As it is difficult, however, to maintain such a temperature, the boil-

* These cuts are taken from Professor Tetmajer's Communications for 1893.

ing test is to be preferred, using the least fire which will maintain this temperature. At the end of this three-hour period the specimens will be found to be extremely hard and solid, like stone. No other test for soundness need be employed. This test is to be employed only with Portland cements, as probably few natural cements would stand it. No satisfactory test for the soundness of natural cements has been found, and the fact that these cements, which may go all to pieces in the boiling test, still stand well in service forms a strong argument against the drawing of adverse conclusions from this test when applied to Portland cements. The question of what tests to apply to determine the weathering qualities of cements is as yet unsolved.

TESTING THE STRENGTH OF CEMENT.

315. Tensile Test Sufficient.—Although cement-mortar and concrete are commonly subjected to a compressive stress only, and hence the strength in compression is of the greatest importance, the only test of strength usually

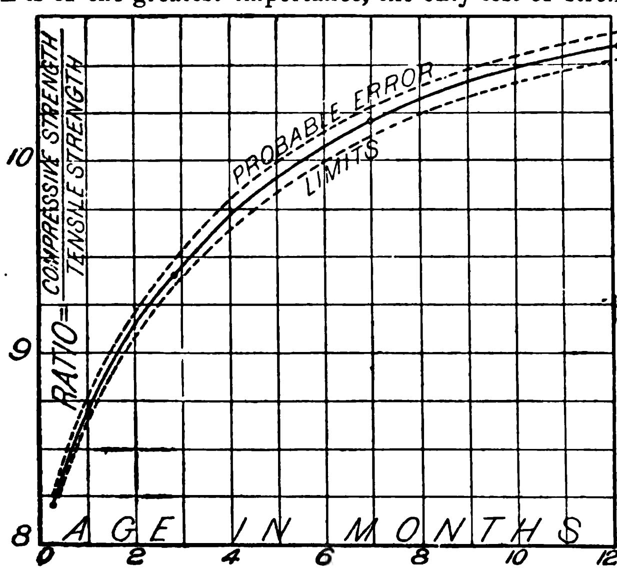


FIG. 337.—Showing the Ratio of the Tensile to the Compressive Strength of Portland-cement Mortar, 1 C.: 3 S., by Weight. Each point plotted is the average of 550 tests of each kind. Equation of curve, $R = 8.64 + 1.8 \log A$. (Data taken from Tetmajer's Communications, vol. VI.)

made is that in tension. In Art. 20 it was shown that the strength of such materials in compression is really their strength in shearing, and for a granular material the strength in shearing would be expected to vary with the strength in tension. It was to be presumed, therefore, that the tensile strength of cement would have a definite relation to its compressive strength. The author is now able to establish this relation as shown in Fig. 337. Here 55 samples of Portland-cement mortar, one of cement to three of sand, by weight, were tested by Professor Tetmajer both in tension and in

compression, there being fifty tests of each kind from each sample. One third of these were left to harden in air, and two thirds hardened under water. These fifty tension- and fifty compression-test specimens of each mixture were divided into five lots of ten each, and these were tested in five periods of time, namely, in 7 days, 28 days, 84 days, 210 days, and 1 year. The average ratios of the compressive to the tensile strength of the 550 tests of each kind made at each of the above periods are plotted in Fig. 337, and joined by the full line. The probable error of each of these ratios was also determined from the residuals obtained by comparing each of the fifty-five results with its average, and these probable error-limits are also indicated in the diagram. These limits are so uniform and so small as to lead to the necessary conclusion that the ratio between the compressive and the tensile strength of cement is a very rigid one for any given age, but that it increases with the age of the mortar. This curve is very nearly represented by the following equation, the maximum deviation of which from the observed locus is less than one half of one per cent

$$\text{Ratio}: \frac{\text{Compr. strength}}{\text{Tensile strength}} = 8.64 + 1.8 \log A,$$

where A = age of the cement-mortar in months. The compression tests were made upon cubical forms. The value of this study is not so much the determination of the true relation between the tensile and the compressive strength of cement-mortar as it is to show that the tensile test is sufficient to determine compressive strength.*

316. Standard Consistency of Neat-cement Test-specimens.—It has been found impracticable to specify any particular percentage of water for all kinds of cement, or even for all brands of one class, as of Portland cement, or of natural cement, or of slag-cement. A certain consistency of the gauged cement demands various percentages of water with different brands of the same class of cements. It is necessary, therefore, to have a standard method of fixing this consistency. The effect of using varying quantities of water with a single brand of cement is shown in Figs. 338 to 343. When an excess of water is used the briquettes are greatly weakened for short periods, but the effect partly disappears with time. When too small a quantity of water is used, it requires too much work to thoroughly compact the briquettes, and the results are apt to be irregular.

The French Commission have adopted a modified form of Prof. Tetmajer's method of determining consistency, which is as follows:

(1) Take one kilogram (2 lbs. 5 oz.) of cement, place it on a marble slab, arrange it in a crater-like form, and add at one pouring all the water which is to be

* M. Feret has shown in *An. d. Ponts et Chaussées*, 7th series, vol. IV. p. 1, Fig. 19 (1896), that this ratio becomes greater for higher proportions of sand. In fact the compressive strength varies uniformly with the proportion of cement used, while the tensile strength is nearly constant for small proportions of sand but falls rapidly for the poorer mixtures.

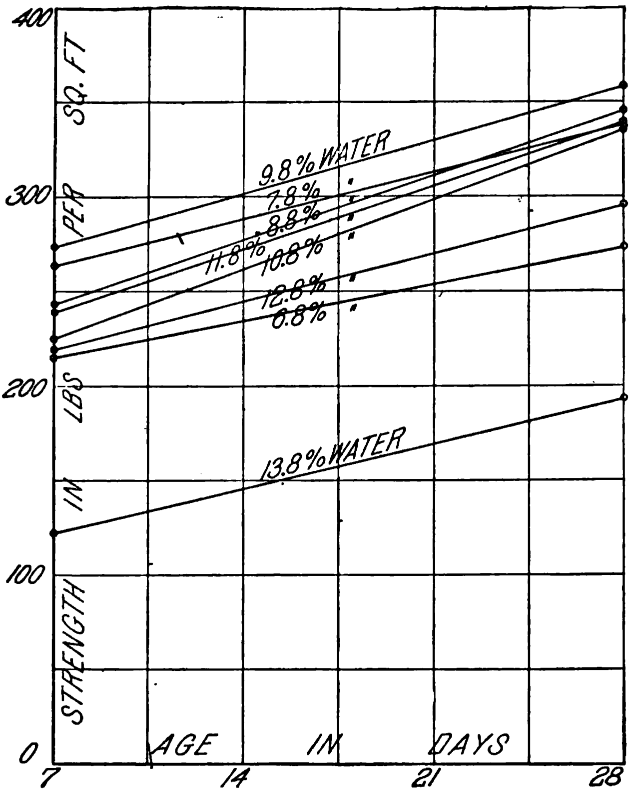


Fig. 338.—Effect of Varying Percentages of Water used in Gauging Portland-cement Mortar, 1 C. : 3 S. Average results on five brands of cement. Each point plotted is the mean of fifty tests. (Tetmajer, vol. VII, 1894, p. 10.)

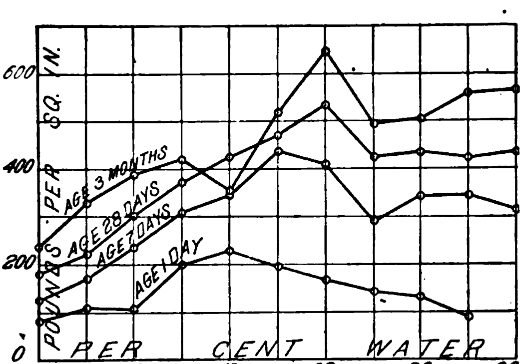


Fig. 339.—Effect of Varying Percentages of Water in Gauging Neat Portland Cement. (*Jour. West. Soc. Engrs.*, vol. I. p. 82, Table XVIII.)

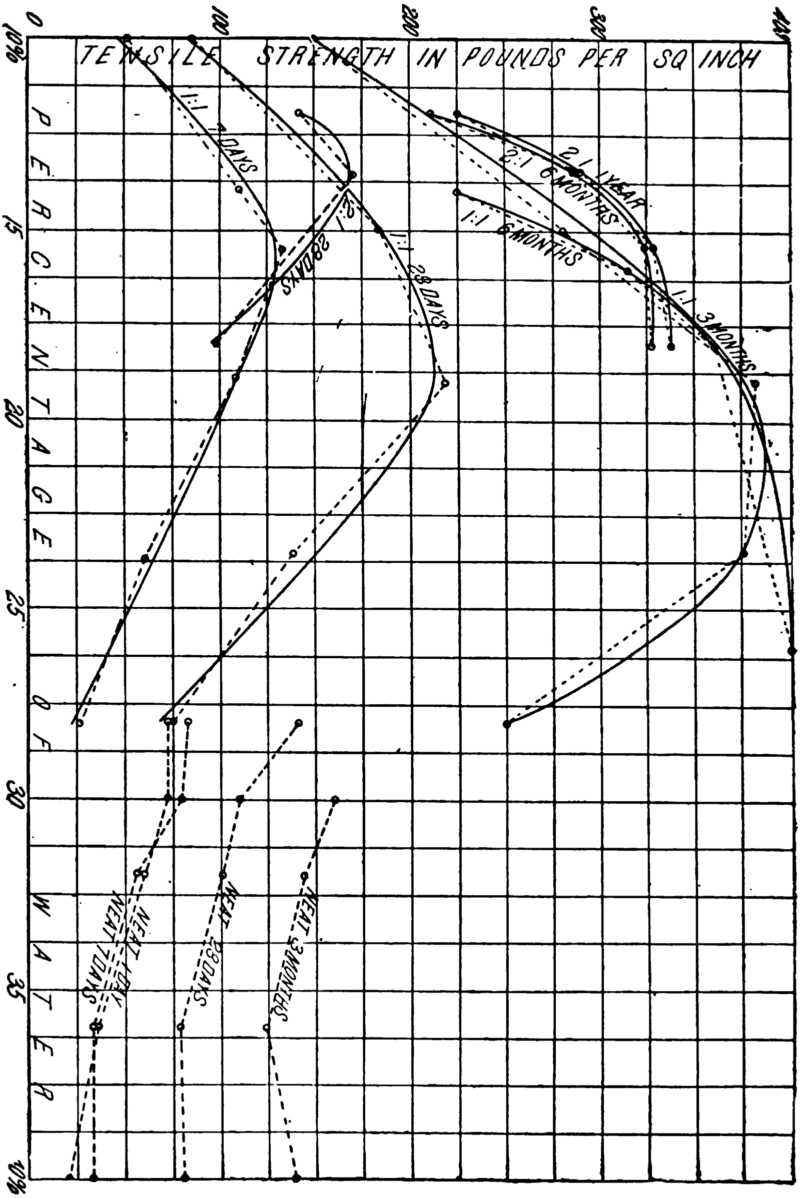


FIG. 340.—Tensile Strength of Natural Cement-mortars, mixed with Different Percentages of Water. (Wheeler, *Rep. Chf. Engrs., U. S. A.*, 1894, p. 2332.)

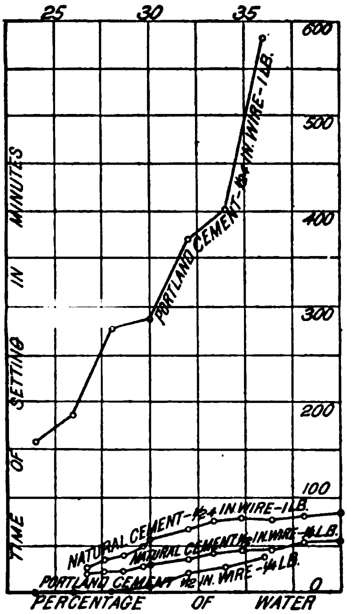


FIG. 341.—Effect of Varying Percentages of Water on Time of Setting of Neat Cement. (Wheeler, *Rep. Chf. Engrs.*, 1895, p. 2935)

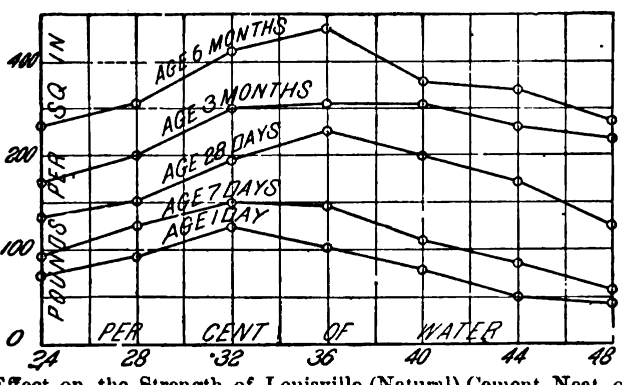


FIG. 342.—Effect on the Strength of Louisville (Natural) Cement, Neat, of a Varying Percent of Water in Gauging. (*Jour. West. Soc. Engrs.*, vol. I. p. 82, Table XVI.)

used, this volume being that necessary to satisfy the conditions described in (2). The water to be either fresh or salt, as may be specified. The whole is then stirred and turned rapidly with a trowel for *five minutes*, counting from the instant the water was added.

(2) With a portion of this gauged cement fill a vessel having an interior form of a truncated cone, 8 cm. (3¼ inches) in diameter at bottom, 9 cm. (3⅝ inches) in diameter at top, and 4 cm. (1⅝ inches) deep, smoothing it off quickly on top with the trowel.

Upon the centre of this top surface bring to bear normally and slowly a cylinder of polished metal 1 cm. (⅜ inch) diameter and weighing 0.3 kilogram (11 oz.), having a full, flat, transverse sectional base. The apparatus to be constructed so as to

indicate the thickness of the film of mortar remaining below the cylinder when it ceases to settle under its own weight. Two tests to be made on the same cake.

The consistency to be considered as normal when the cylinder stops just ¼ inch from the bottom of the cake.

For quick-setting cements use one half the amount of dry cement, and mix one minute instead of five.

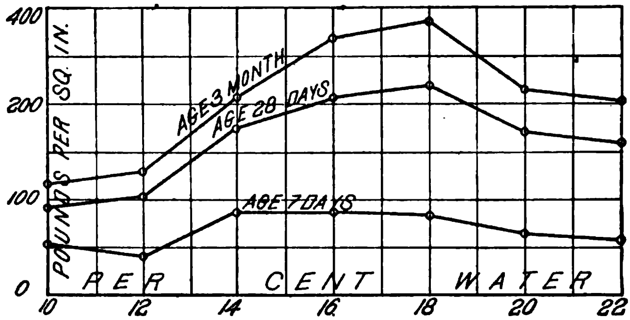


FIG. 343.—Effect of Varying Percentages of Water in Gauging Utica (Natural) Cement-mortar, 1 C. : 1 S. (*Jour. West. Soc. Engrs.*, vol. I. p. 82, Table XV.)

317. Normal or Standard Sand.—That the quality of the sand exerts a marked influence on the strength of cement-mortar is shown by Figs. 344 and 345. These tests show the great superiority of calcareous over siliceous

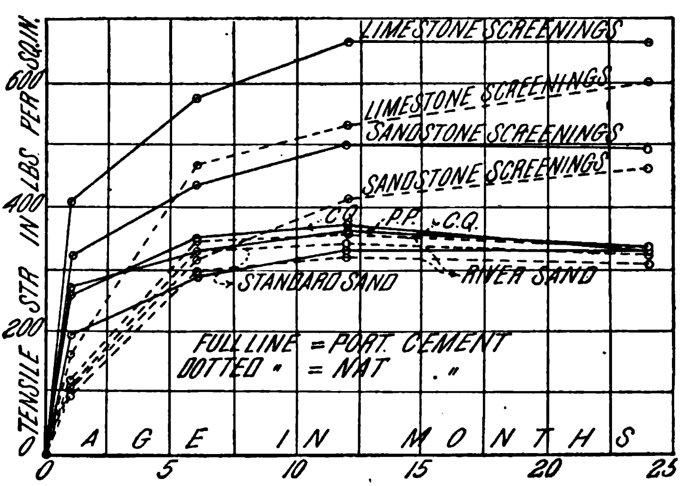


FIG. 344.—Effect of the Quality of the Sand on Strength of Cement-mortar, 1 C. : 3 S. (Wheeler, *Rep. Chf. Engrs., U. S. A.*, 1894, vol. IV. p. 2321.)

sands in giving strength to the mortar. Evidently, however, sands containing small shells containing air-spaces should be excluded.

To find the composition of a sand immerse it in cold hydrochloric acid, which will dissolve the ~~siliceous~~ portion. The residuum may then be separated into the insoluble ~~calcareous~~ sand and the clay, by rubbing and wash-

ing, and thus its three significant constituents determined with sufficient accuracy for commercial purposes.

Not only the strength but the permeability of mortar depends on the size of the sand-grains; and as the resistance to the decomposing action of

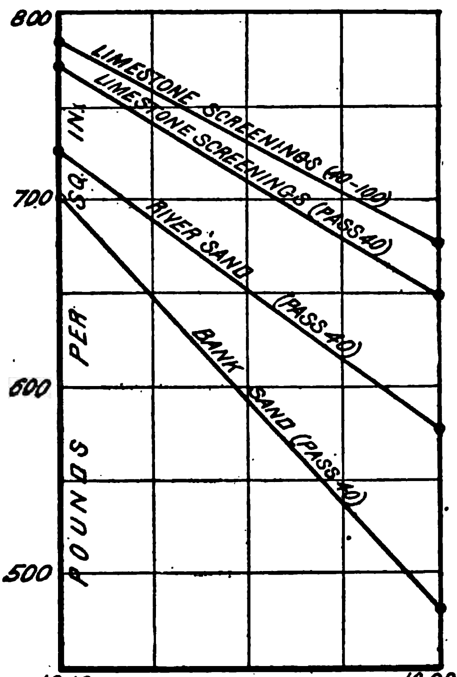


FIG. 345.—Comparative Value of Different Sands in Portland-cement Mortar 18 months old (in water). (Wheeler, *Rep. Chf. Engrs.*, 1895, vol. IV. p. 2958.)

frost and sea-water depends almost wholly on its impermeability, the life of the mortar, in exposed situations, is largely dependent on the character of the sand used.

In order that tests of the strength may be comparable, therefore, it is necessary to choose a normal, or standard, sand. In Germany and in France natural sands are chosen, while in the United States a committee of the American Society of Civil Engineers recommended in 1885 the use of crushed quartz, such as is used in making sand-paper, of a size which passes a No. 20 sieve and is stopped on a No. 30 sieve. This leaves all the grains with maximum dimensions of from 1 mm. to 1.5 mm. When the grains are so nearly of the same size and very angular or "splintery" the proportion of voids is very great, so that a mixture of 1 cement to 3 sand by weight will not be solid without a great amount of pounding on the briquette, which must be mixed dry to enable it to receive such treatment.

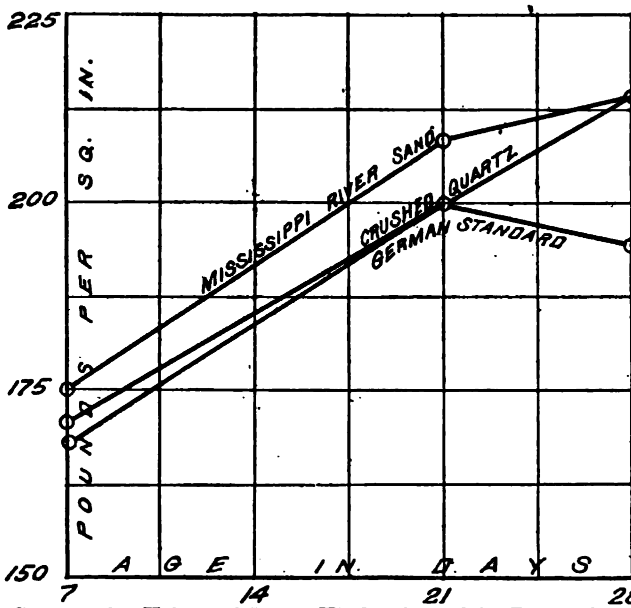


FIG. 346.—Comparative Value of Three Kinds of Sand for Portland-cement Mortar, 1 C.: 3 S. (*St. Louis Water Dept.*, 1895.)

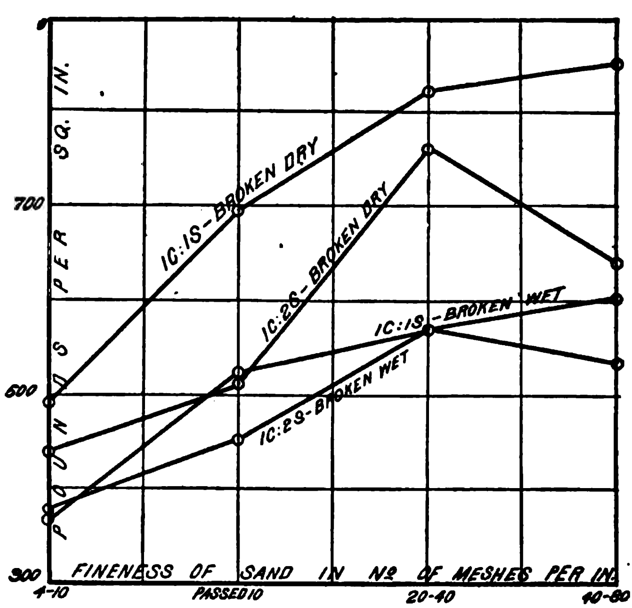


FIG. 347.—Effect of Fineness of Sand on Strength of Portland-cement Mortar, 1 : 1 and 1 : 2. Age 6 months. (Wheeler, *Rep. Chf. Engrs.*, 1895, p. 2972.)

The French Commission have adopted the following:

Normal or standard sand consists of sand found on the beach at Leucate, and is of three sizes:
No. 1, passing a sieve of 1 mm. and retained on one of 0.5 mm. mesh.
No. 2, passing a sieve of 1.5 mm. and retained on one of 1 mm. mesh.
No. 3, passing a sieve of 2 mm. and retained on one of 1.5 mm. mesh.

Simple normal sand is construed as meaning No. 2. *Composite* or *mixed normal sand* is construed as meaning all three sizes in equal parts, this mixture approaching closely the sand ordinarily employed in engineering works. The finest grade, No. 1, corresponds to such fine sand as is found in the sand-dunes along our sea and lake shores, while the coarsest, No. 3, corresponds to the very coarse sand taken from the bed of a rapidly-flowing river. The composite or mixed sand is exclusively used in standard tests of mortar. This is to be commended, as it gives fewer voids, and a mixture of 1 cement to 3 sand readily makes a perfectly solid test specimen. The above sieves, having meshes of 0.5, 1.0, 1.5, and 2 millimeters, would be found to have approximately 35, 20, 15, and 11 meshes per inch respectively.

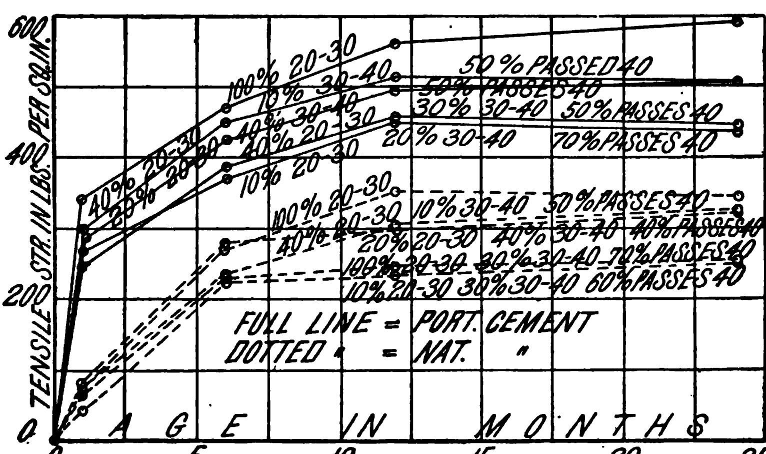


FIG. 348.—Showing Effect of Varying Fineness of Clean River-sand in Cement-mortar, 1 C. : 3 S. (Wheeler, *Rep. Chf. Engrs.*, vol. IV, 1894.)

In place, therefore, of using a 20–30 sand in making up standard mortar-test specimens, as has become customary in America, in accordance with the American Society of Civil Engineers Committee's recommendation, the French are using a sand composed equally of three grades, which are respectively 11–15, 15–20, and 20–30 sieve samples. This gives sand-grains varying from 0.5 mm. to 2.0 mm. in size, or a variation in size of 300% of the smallest, while the American Society of Civil Engineers' standard allows

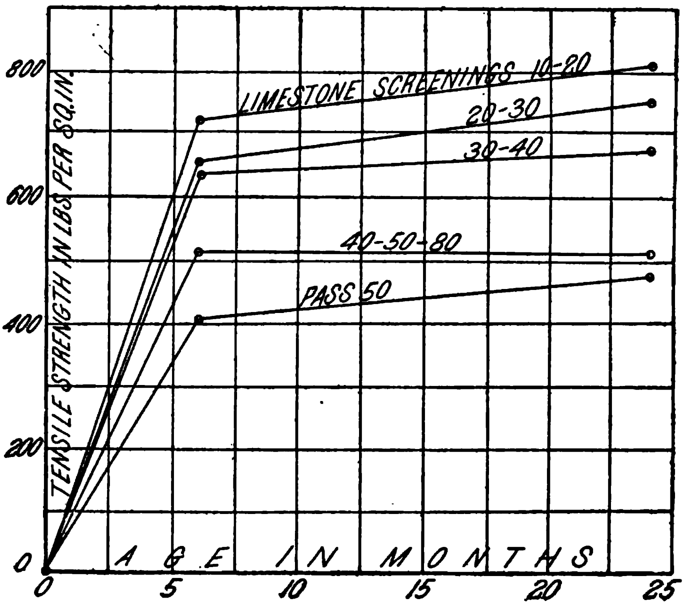


FIG. 349.—Effect of Size of Limestone Screenings when used as Sand in Portland-cement Mortar, 1 C. : 3 S. (Wheeler, *Rep. Chf. Engrs.*, 1894, vol. IV.)

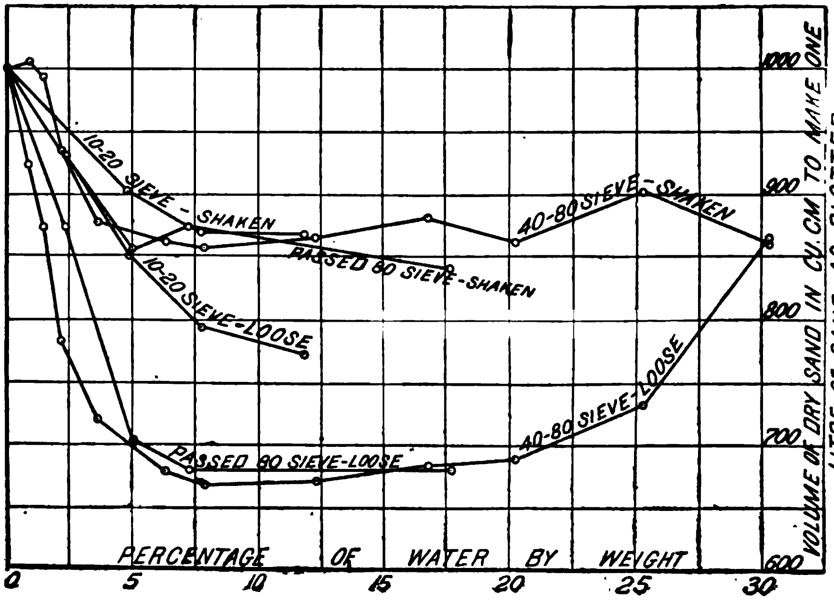


FIG. 350.—Variation in Volume of Different Grades of Sand by the addition of small quantities of water. (Wheeler, *Rep. Chf. Engrs.*, 1895, p. 2935.)

but 50% variation in the size of the sand-grains for the standard mortar-tests.

318. Standard Consistency of Cement-mortars.—The standard cement-mortar is composed of one part of cement to three parts of standard sand, by weight. The great variation in volume of sand with varying percentages of water, as shown by Fig. 350, precludes the volume measurement even of the sand, while with the cement there is no fixed relation between volume and weight. It has been customary for many years in Germany to use the minimum amount of water which would enable this mixture to be compacted in a briquette by the action of what is known as Böhme's hammer (see Fig. 352), in the use of which 150 blows is given to each briquette. As this is a condition very far removed from those of actual practice, it has always been objected to in other countries, and has never come to be standard in America. When a greater quantity of water is used, however, so as to give a plastic mortar, it cannot be compacted by pounding and it becomes more difficult to obtain uniform results. The French Commission have studied this question most effectually, and, while they are forced to still recognize the dry mixture as above described, they strongly recommend the use of plastic mortars, and express the hope that the German standard method of preparing these specimens will fall into disuse. Their recommendations on this subject are as follows:

1. Standard plastic cement-mortar shall be composed of one part of cement (250 grams) to three parts of mixed normal sand (750 grams), this being composed of equal parts of numbers 1, 2, and 3, as described in Art. 317. These will be mixed thoroughly before water is added, on a marble slab, and then gauged with the full quantity of water, either fresh or salt as the case may be, and vigorously stirred and worked for five minutes.

The quantity of water to be used to be such that when the vessel described in Art. 316 is filled with the mortar and smoothed off, a few strokes of the trowel upon the sides of this vessel will cause the mortar to liquefy slightly at the surface.

For cements which set rapidly the total quantity of materials used to be reduced to 500 grams, and the gauging to be continued for one minute instead of five.

2. Standard dry-cement mortars shall be composed of one part of cement (250 grams) to three parts (750 grams) of standard sand No. 2 (described in Art. 316), these to be mixed while dry on a marble slab, and then an amount of water added equal to one sixth of that necessary to use in bringing one kilogram of the same kind of neat cement to the standard consistency described in Art. 316 plus 45 grams additional.*

3. If other proportions are desired than one of cement to three of sand, it is recommended that one of cement to two of mixed standard sand, and one of cement to five of mixed standard sand, be used; these also to be regarded as standard, rich, and poor mortar, respectively. The amount of water to be used in each case to be such as to produce a plastic mortar which will satisfy the conditions named above in 1.

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* For the standard dry mortars of varying proportions of sand, and for all kinds of cement, the amount of water to use was found to be, in grams for 1 kg. of the dry mixture, $w = \frac{1}{6} WC + 45$, where W = weight of water in grams required to bring 1 kg. of the pure cement to the normal consistency described in Art. 316, and C = weight in kilograms of the cement entering into the dry mixture.

In place of the hand-mixing on a slab, as described above, the author has used with very satisfactory results the Faija mechanical mixer, made by Riehlé and shown in Fig. 351. Something of this sort is especially helpful in the case of sand mixtures, the sand and cement being first mixed dry and then from three to five minutes after wetting.

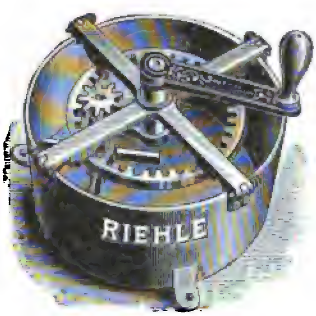

Fig. 351.

The St. Louis Water Department use for neat cement a "jig," consisting of a pair of cups mounted vertically on a reciprocally moving head-piece, operated by a very rapid circular motion, like the familiar "milk-shake" apparatus. (See drawings and description in *Engr. News*, vol. xxv. p. 3, 1891.)

319. The Formation of the Briquettes.—The following rules for forming the briquette are based largely on the Report of the French Commission, but they also fairly represent the best current American practice.

A. *For Standard Plastic Mortar*, 1 *Cement to* 3 *Sand.*

(1) The briquette to be of the form shown in Fig. 355 or Fig. 356, having just one square inch of minimum cross-section.*

(2) The moulds to be quite clean, and to be rubbed with an oiled or greased linen cloth, and placed on a plain marble slab, or plate-glass, or polished metal surface. Six moulds to be simultaneously filled to overflowing (if the cement is slow-setting, and but four moulds to be filled if it is quick-setting), the entire amount required for one mould to be inserted at one time. The mortar to be pressed into the moulds with the fingers, and a few strokes given to the side of the mould with the trowel. This having been done for the entire set (of six or four as the case may be), the excess of mortar is carefully removed with a straight-edged blade resting on the top edge of the moulds, but without exerting any compression on the material below this plane. The surface is then polished off with the trowel, and the whole covered with wet cloths, and kept from sun and wind, in a saturated atmosphere, and at a temperature of from 60 to 70° F. When making plastic briquettes of neat cement, it may be best to allow them to stand a while before removing the excess of material and polishing off.

(3) After the mortar has set (at the end of 24 hours, or sooner) the mould is tapped lightly on the side to loosen the briquette from the bed-plate, when the mould is unlocked and removed from around the briquette. These are not raised from the plate (if the moulds are removed inside of 24 hours), but are covered with wet cloths until 24 hours have elapsed from the time of mixing with water.

* The European standard section is 5 sq. cm. or 0.8 sq. in. The ordinary American form of briquette (Fig. 354) should be abandoned at once, since it is impossible to prevent such briquettes of neat Portland cement from breaking in the clips.

For very quick-setting cements the time period in air for neat cement may be reduced to one hour, and for mortar briquettes to three hours.

A careful weighing of the briquettes when removed from the moulds gives a very good check on the uniformity of their composition.

(4) At the expiration of the period described in (3) the briquettes are placed in their required medium till tested. If they are placed in fresh water, this should be changed as often as once a week. If placed in sea-water, it should be changed every two days for the first week, and then once a week. The water-volume should be at least four times that of the briquettes immersed in it.

If the briquettes are to harden in air, this should be kept near the point of saturation, and they should be protected from all air-currents and from the rays of the sun. The temperature of the medium, whether of air or water, should remain from 60° to 65° F. (15° to 18° C.).

(5) The tensile testing-machine to be so arranged as to give a uniform imposition of the load at the rate of 12 pounds (5 kg.) per second. The form of the grips to be that shown in Fig. 363.

(6) Standard tests of cement-mortar to be made at the end of 7 days, 28 days, 3 months, 6 months, 1 year, and 2 years, all computed from the time of gauging. For mortar made from quick-setting cement the shortest period to be 24 hours, and for quick-setting neat cement briquettes the short periods to be 3 hours, and 24 hours from the time of gauging.

(7) So far as possible the six briquettes made from a given gauging to be divided uniformly among the lots to be tested at different periods. Thus if tests are to be made after six such periods, as named above, then one briquette from each gauging to be assigned to each period.

A single result for any period to be the mean of the tests on six briquettes, defective samples to be rejected, however, and the mean to be derived from the remaining perfect tests, all the facts to be indicated on the record.

The results to be given as so many pounds per square inch (kilograms per square centimeter) tensile strength on the standard form of briquette of one square inch (5 sq. cm. in Europe) in cross-section.

B. *For Standard Dry Mortar*, 1 *Cement to* 3 *Sand.*

(1) All the conditions specified in A to be complied with. In addition to these the following rules will be observed:

(2) At the moment of mixing, the cement, the sand, the water, and the air to be at a temperature between 60° and 65° F. (15° to 18° C.). After the moulds are filled to overflowing, and the mortar has been pressed to place with the fingers, it will be pounded on the surface with a heavy spatula, 14 inches long over all, including the handle, and having a surface of blade of four square inches (25 sq. cm.) and weighing 9 ounces* (250 gr.). The

* No cut of this is given. If the blade itself is to weigh 250 gr., it would be, say, one inch wide, one-half inch thick, and four inches long.

briquette to be beaten at first with light strokes near the ends, then towards the centre. These to be followed by heavier strokes, always following the same course over the surface of the briquette, and continuing this treatment till the mass becomes somewhat plastic and water begins to appear at the surface. The surface is then scraped and smoothed off as before.

For many years standard cement-mortar briquettes have been formed in Germany almost exclusively by the use of a machine shown in Fig. 352,

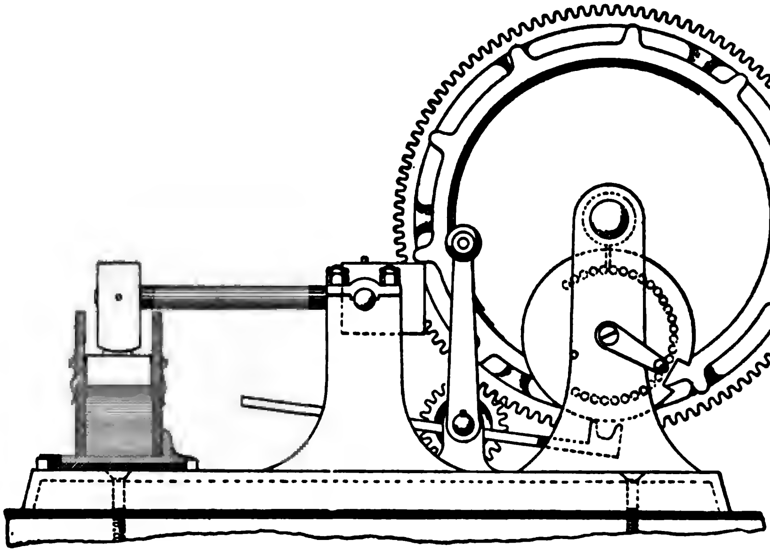

FIG. 352.—Dr. Böhme's Hammer for making Cement Briquettes.

which is the invention of Prof. Böhme of Charlottenburg. The hammer is driven by a wheel with ten cams, connected by simple gearings with a crank and handle. The steel hammer weighs four and one-half pounds. This apparatus may be used for making either tension- or compression-test specimens, and is preferably used when these are made of standard mortar mixed dry. There is an automatic stop which acts at the end of 150 strokes, this being the usual number of blows given to each test-specimen. Fig. 353 shows an apparatus used by Prof. von Tetmajer and which has now been recommended for general use to the Fifth International Convention for Unifying the Methods of Testing Engineering Materials, which met at Zurich in September 1895. The French Commission have not included either of these kinds of apparatus in their standard specifications given above.

320. The Form of the Briquette.—After nearly a half-century of experimenting on a great many different forms of briquettes, two leading forms are now used to the practical exclusion of all others. The English form shown in Fig. 354, having a minimum section of one square inch, is used in England and in America, and the German form shown in Fig. 355, having a minimum

section of five square centimeters, is used on the continent of Europe, having recently been recommended by the French Commission.

A great objection to the English standard form shown in Fig. 354 is that a very large proportion of briquettes of neat cement over four weeks old (about 50 per cent) break in the clips and not on the minimum cross-section.

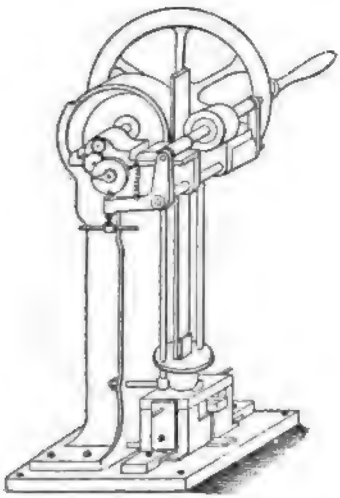

Fig. 353.—Tetmajer's Apparatus for Compacting Dry-mortar Briquettes, with an adjustable height of drop. (*Fr. Com. Rep.*, vol. I. p. 287, and also *Zurich Laboratory Communications*, vol. VII. p. 118.)

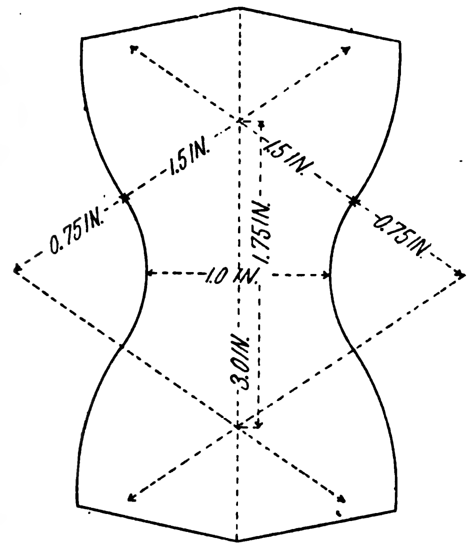

Fig. 354.—Standard Form of Briquette used in England and America. Full size.

This is partly the fault of the small bearing-surface provided in the form of clips used, but it is also largely due to the form of the briquette. The angle which the two bearing-surfaces form with each other is small, and the compressive stress resulting is correspondingly large, so that the briquette is apt to fail in the plane of the bearings from a combined vertical tension and a lateral compressive stress.

The conditions which should be fulfilled in the form of a cement briquette for tensile tests are:

(1) The bearing-surfaces in the clips should form an angle with each other of more than 90°.

(2) The minimum section should be removed far enough from the plane of the bearings in the clips to insure a nearly even distribution of stress over this minimum section.

(3) The minimum section should be small enough to insure rupture on this portion of the briquette, but the reduction should be by gentle curves.

The English form (Fig. 354) is very defective in the first requirement, as

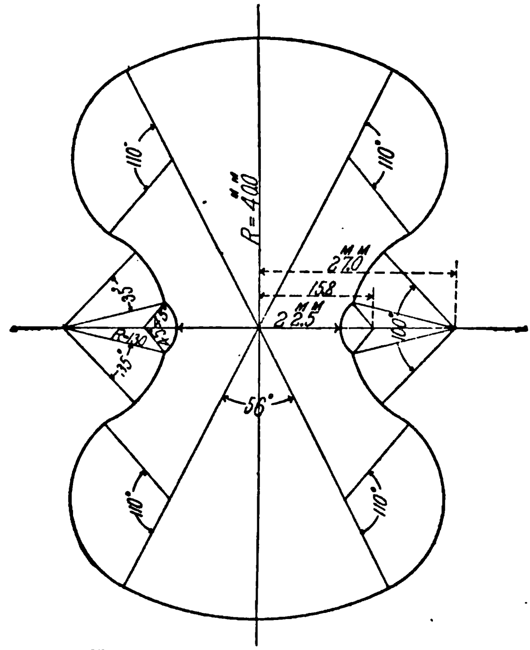

FIG. 355.—The European Continental Form of Briquette.

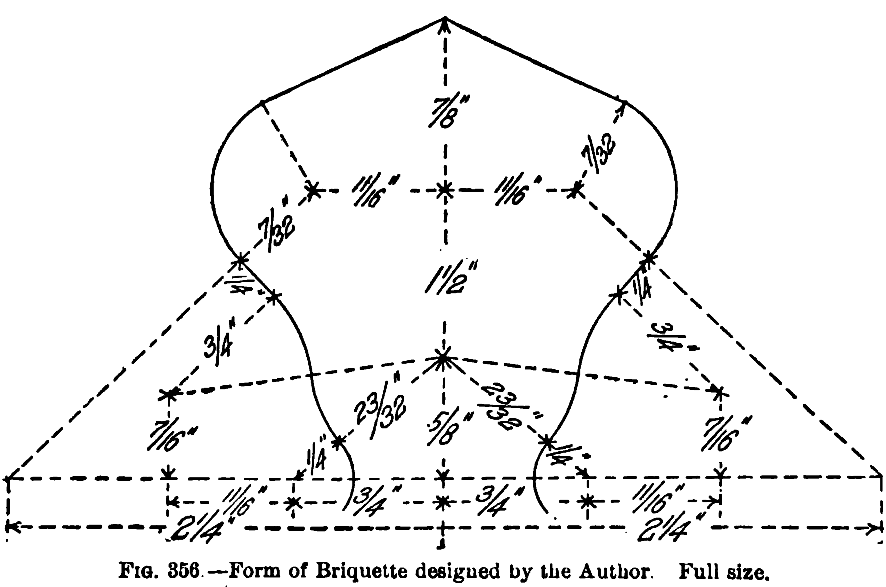

FIG. 356.—Form of Briquette designed by the Author. Full size.

this angle is only about 60°, while the German form in this respect is excellent, its angle being 100°.

Both forms are defective in the second requirement, as they are shortened up from reasons of economy and convenience.

The German form, Fig. 355, is defective as to the third requirement, in that the reduction of section is too sudden.

The author has devised and used a modification of the German form, as shown in Fig. 356, and he finds that it gives over twenty-five per cent greater strength than the English form, and the briquettes never break in the clips he uses with it, shown in Fig. 363. The greater strength of this form results from the more even distribution of stress across the section of rupture, owing to the farther removal of the bearing-surfaces of the clips, while the large angle formed by these surfaces is maintained. While this of necessity makes a much larger briquette, it would seem to be the only way to secure a form which will develop the real strength of the material.

321. To Find the Distribution of Stress over the Minimum Section of a Cement Briquette.—The following is a development of the theory of the distribution of stress over the minimum section of a cement briquette, published by M. Durand-Claye in the *Annales des Ponts et Chaussées* in June 1895.* Let M and N, Fig. 357, be the points of application of the external

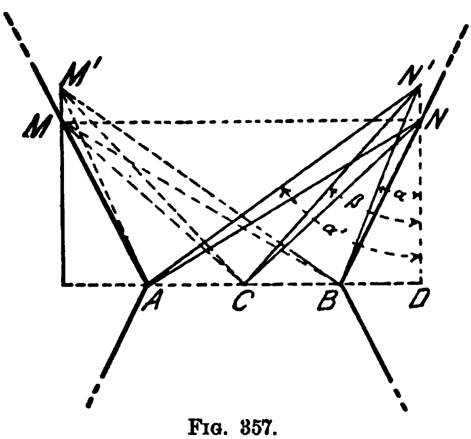

FIG. 357.

forces applied to the briquette, which is here given a somewhat conventional form. As a result of the application of the vertical forces at M and N these points are raised to M' and N' with respect to the fixed axis AB, through the distortion of the specimen. The original lines NB, NC, and

* Prof. Aug. Föppl (Bauschinger's successor) has now shown that a greater tensile stress is actually developed on the outer fibres of stone beams than can be obtained on tension specimens, and he thinks this is because of the uneven distribution of stress over the cross-section of the tension-test specimen. (*Communications from the Munich Laboratory*, vol. XXIV, 1896.)

NA of the specimen now become $N'B$, $N'C$, and $N'A$, and the stresses along these lines are proportional to the deformations $N'B - NB$, $N'C - NC$, and $N'A - NA$, respectively. Similar relations exist on the corresponding lines drawn from M. The total stress at the points A, C, and B, therefore, will be equal to the sum of the vertical components of the stress at each point arising from the external forces at the two points of application M and N.

Let α, β, and α' represent respectively the angles which the three lines from N to B, C, and A form with the vertical.

Now the stretch of the lines ND, NB, NC, and NA due to the displacement NN' is as 1, $\cos \alpha$, $\cos \beta$, and $\cos \alpha'$, respectively, and the proportional stretch of a line is the total stretch divided by the length of the line; hence the proportional stretch of these lines is as 1, $\cos^2 \alpha$, $\cos^2 \beta$, and $\cos^2 \alpha'$, respectively.

But the stress is as the proportional stretch; hence the stresses along the lines NB, NC, and NA are as the squares of the cosines of their respective angles with the vertical. Since we are only concerned with the vertical components of these stresses, and since these are respectively equal to the inclined stress into the cosine of the same angle, the vertical stresses at B, C and A due to the external force at N are to each other as $\cos^3 \alpha$, $\cos^3 \beta$, and $\cos^3 \alpha'$.

The total vertical stress at each of these points, however, is the sum of the two stresses arising from the external forces at both M and N; hence it follows that if we represent by R the total vertical stress at A and B, and by r_0 the total vertical stress at C, we have

$$\frac{R}{r_0} = \frac{\cos^3 \alpha + \cos^3 \alpha'}{2 \cos^3 \beta}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

By trial the law of the distribution of stress over the section AB, by equation (1), is very nearly that of a parabola. Making this assumption, and knowing that the area of the exterior portion of the rectangle enclosing a parabolic segment is one third that of the rectangle, we have, for the mean stress over this section,

$$r = \frac{P}{S} = r_0 + \frac{1}{3}(R - r_0) = \frac{1 + 2\frac{r_0}{R}}{3}R, \quad . \quad . \quad . \quad . \quad (2)$$

where P = total breaking strength of the briquette and S = its sectional area.

For the three forms of briquette shown in Figs. 358, 359, and 360 the values of $\frac{R}{r_0}$, from eq. (1), are 1.22, 2.04, and 2.12, and the values of the average stress $r = \frac{P}{S}$ are, from eq. (2), 1.14, 1.52, and 1.54, respectively.

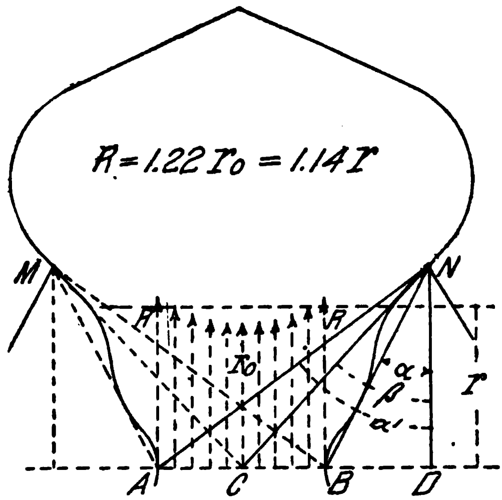

Fig. 358.—Showing the Distribution of Stress in the Author's Form.

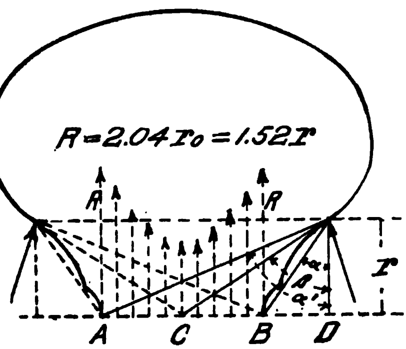

Fig. 359.—Showing the Distribution of Stress in the German Form.

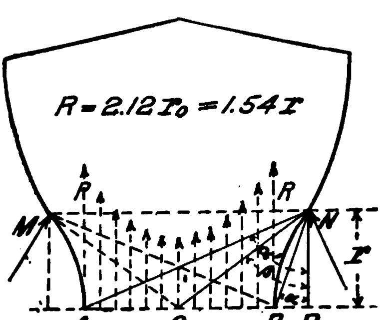

Fig. 360.—Showing the Distribution of Stress in the Standard English and American Form.

The last of these forms (Fig. 360) is the standard form employed in England and America, and is that recommended by the Committee of the American Society of Civil Engineers. The second (Fig. 359) is the standard form employed on the continent of Europe, and is commonly spoken of as the "German standard." The first (Fig. 358) is a form devised by the author to give a more even distribution of stress across the minimum section. An extended series of tests by the St. Louis Water Department shows results on neat Portland cement over 25 per cent greater on briquettes of the form shown in Fig. 358 than was obtained on exactly similar briquettes of the common American form shown in Fig. 360.* These higher results are partly due to the improved form of clips shown in Fig. 363.

322. The Form of the Moulds.—It is customary to use single moulds, though multiple or gang moulds are often used where great numbers of briquettes are to be made. In either case they should be made in two parts, as shown in Fig. 361, so as to be easily removed after the samples have set without danger of breaking the test-specimens. The parts may be held together by a spring, by clamps, or by a latch. They should always be oiled or soaped before using, to prevent the cement from adhering to them.

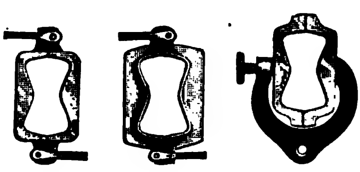

FIG. 361.

Where the English system of measures is used it is now common to make the minimum cross-section of the briquette 1 inch square. It has been thought that the strength varied with the size of the section, as is apparently proved by the results plotted in Fig. 362, but in all probability this variation can now be explained by the greater inequality in the distribution of the tensile stress over the larger cross-sections. At least it seems fair to assume this to be the case until it has been disproved.

323. The Clips—Their Bearings and Mountings.—Next in importance to the form of the briquette is the character of the clip by means of which the briquette is broken. The essential features of perfect clips are:

1. They must grasp the briquette by a hard-cushion bearing on four symmetrical flat surfaces.

2. They must be freely suspended from a pivot bearing, so as to turn without friction while under stress.

3. They must be so rigid that they will not spread appreciably when subjected to their maximum load.

The first requirement is necessary in order to avoid crushing the

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* If we use the subscripts a and b to distinguish the forms in Figs. 358 and 360 respectively, we have, since $R_a = R_b$, $1.14r_a = 1.54r_b$, or $r_a = 1.35r_b$. That is to say, the form shown in Fig. 358 should be 35% stronger than that shown in Fig. 360.

briquette by the concentration of the load on a line or on a few points. Very hard rubber pieces should be dovetailed into the metal clips at the bearing-surfaces, and allowed to project a little beyond the metal.

When the second requirement is satisfied it is advisable to use an adjusting frame for placing the clips symmetrically on the briquette.

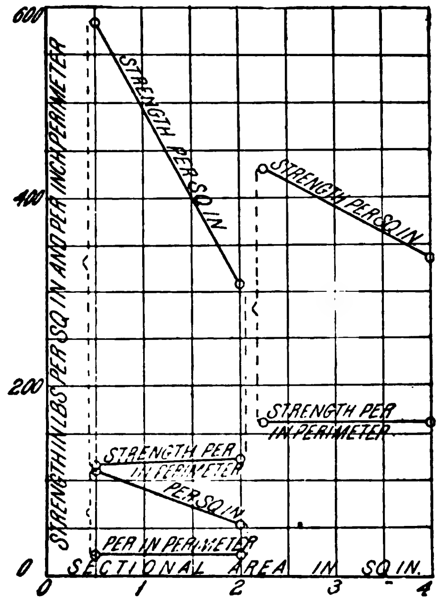

FIG. 362.—Apparent Varying Tensile Strength of Cement for Different Areas of Cross-section of Briquette. (Grant and Whittemore, *Engr. News*, Dec. 14, 1893, p. 468.)

All these demands are well satisfied in the form of clip and adjusting-frame shown in Fig. 363, which were devised by the author to be used with his new form of briquette (Fig. 358).*

These clips are suspended from a steel point, the same as in the German (Michaelis) machine. The hard-rubber (gutta-percha) pieces *R* bear directly on the tangent surfaces of the briquette, and they are brought to a symmetrical position (after the briquette is placed) by means of the adjusting-frame, which is set over the screw-heads at top and over the raised guides at bottom and then slipped downwards to a bearing. The screw-clips at bottom are then turned, when both the briquette-clips are held rigidly against the adjusting-frame and in their true position. The movable clip is then screwed down to a hard bearing on the briquette, and the adjusting-frame removed. It might, however, remain on throughout the test if preferred, and in fact it could be permanently attached to the rear sides of the clips.

* Both the moulds and the clips with their adjusting-frame are made by Mahn & Co., instrument-makers, St. Louis, Mo.

These clips, used on briquettes of the form shown in Fig. 356, greatly increase the breaking strength of neat Portland cement.

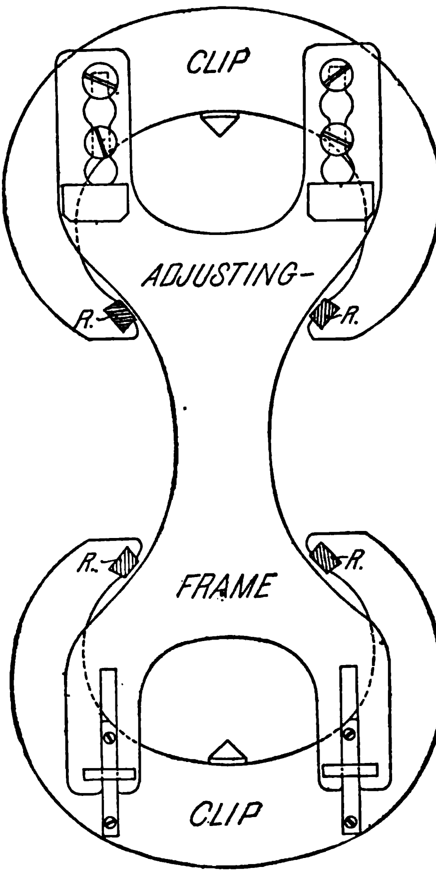

FIG. 363.—The Author's Form of Clip and Adjusting-frame. Half-size. To be used with the form of briquette shown in Fig. 356

324. The Testing-machine.—On the Continent Michaelis' machine, shown in Fig. 364, is almost universally employed. The leverage is 50 to 1, and the load is imposed by means of small shot which escapes from a reservoir into the weight-pan. The dropping of this pan when the specimen breaks shuts off the flow of shot. The pan is then weighed, and its weight, multiplied by 50, (this may be done in the graduation of the scales) gives the strength of the briquette.

A very neat modification of this machine is that made by the Fairbanks Scale Co. and shown in Fig. 365. It is entirely self-contained and dispenses with both the auxiliary reservoir and frame and the weighing-scales. The shot-pan is moved from the end of the weighing-lever and hung from

the hook at the left, and a weight-hook hung in its place on the weighing-beam. The poise is then moved out on this beam, its extreme movement corresponding to a load of 200 pounds. For greater loads "200-pound" weights are placed on the weight-hook and the poise moved out again to balance. In both the machines the load comes on gradually and without

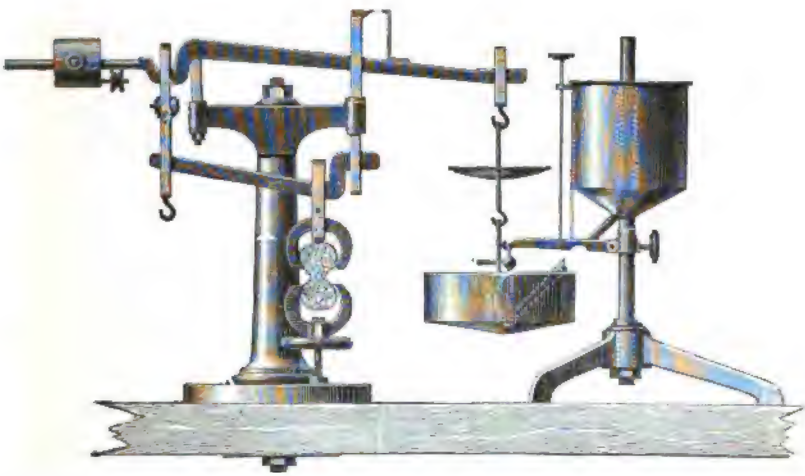

Fig. 364.—Standard Form of German Cement-testing Machine for Tension Tests.

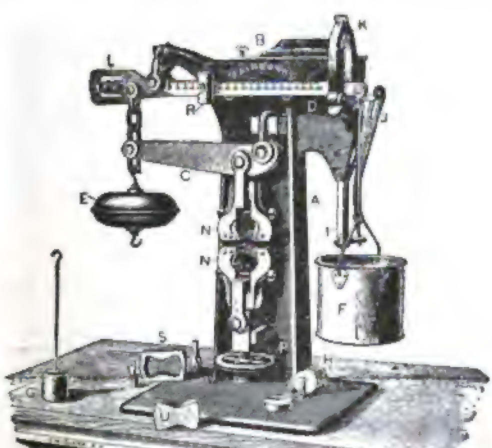

Fig. 365.—The Fairbanks Cement-testing Machine.

shock by the flow of the shot, which is automatically shut off by the dropping of the beam, and the rate of imposition of the load can be regulated by varying the size of the gate. The tightening-screw P is first turned till the weighing-beam moves to its highest limit, and then any further amount to put an initial stress in the specimen short of rupture, after which the

free movement of the weighing-beam is sufficient to break the specimen without any further turning of the tightening-screw. The clips contain adjustable bearings intended to prevent the breaking of the briquettes in the clips.

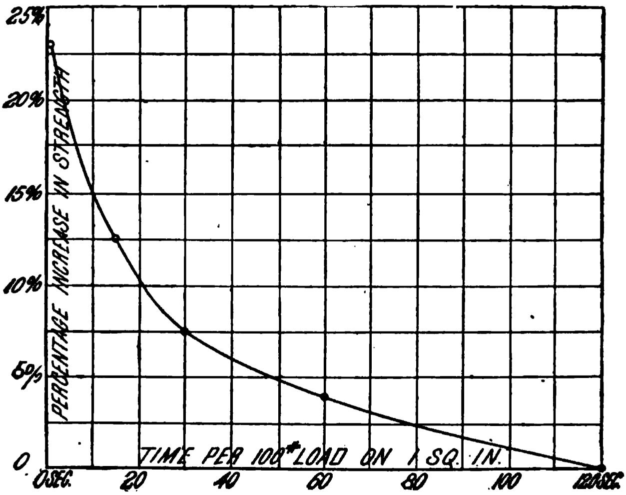

FIG. 366.—Effect of Varying the Rate of Loading on the Tensile Strength of Neat Portland-cement Briquettes. (Faija, in *Trans. Inst. C. E.*, vol. 75.)

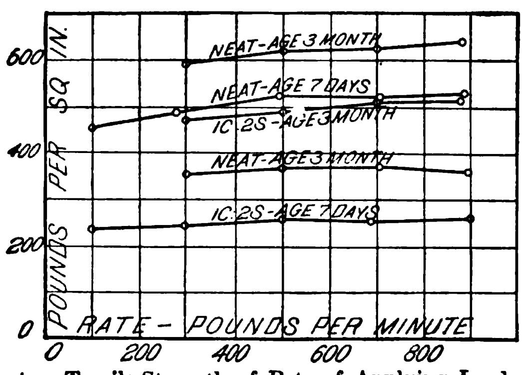

FIG. 367.—Effect on Tensile Strength of Rate of Applying Load. (Wheeler, *Rep Chf. Engrs.*, 1895, p. 2951.)

The machines shown in Figs. 368 and 369 are made by Messrs. Riehlé Bros. and by Tinius Olsen, respectively, both of Philadelphia. They both require the imposition of the load by hand; but as this is through a screw-gear and is very slowly applied, there would seem to be no appreciable unsteadiness in it. The beam is kept in balance at the same time by the same attendant, by moving out the poise till rupture occurs. Evidently the speed here is entirely under the control of the operator, and both these machines give entire satisfaction.

In all these testing-machines the grips or clips are swivelled and mounted in such a way as to allow of a free universal movement, or adjustment of these to the specimen.

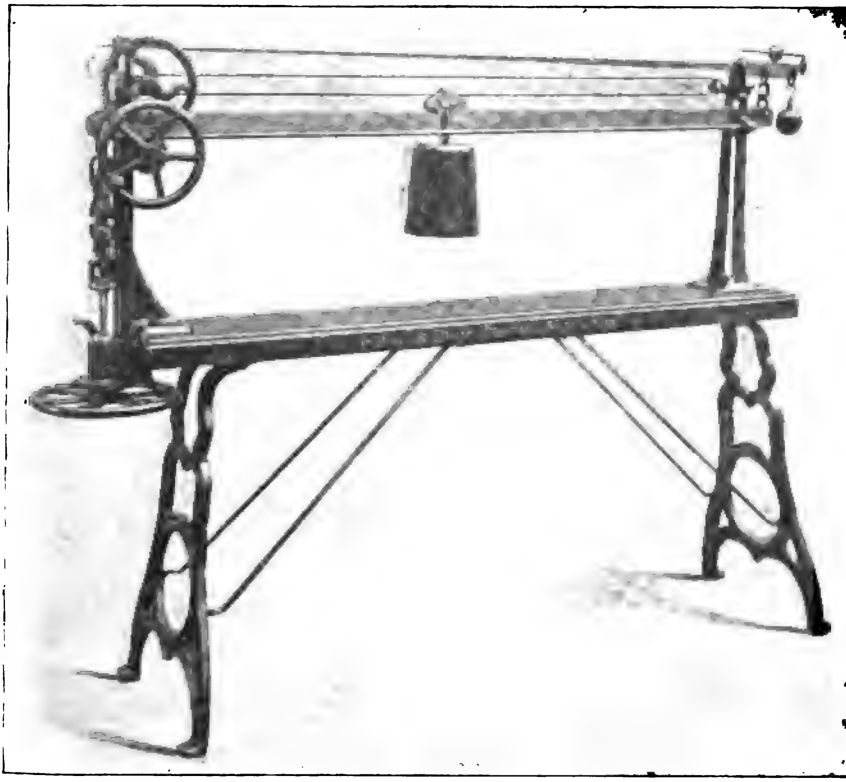

FIG. 368 —Riehlé Cement-testing Machine.

Fig. 370 shows the construction of a cement-testing machine designed and used by Prof. J. M. Porter.*

"The load is applied by water flowing into a tank suspended from the long arm of a very sensitive 15-to-1 lever. The weight of the lever and tank is counterbalanced by an adjustable weight shown on the left. Water is admitted to the tank from a large reservoir on the roof under a practically constant head of 90ft., so that there is no sensible variation of pressure in the stream admitted through a carefully fitted gate-valve in the supply-pipe. The position of this valve at "on," "off," and all intermediate points is shown by an index attached to the stem of the valve and registering on a

* The following description by the inventor is taken from the *Engineering News* of March 5. 1896.

dial marked off with the number of pounds per minute applied to the specimen as determined and verified by previous experiment.

"When the briquettes break, the lever drops a few inches, then the plunger at the right end of the lever enters the pneumatic stop, and the lever and tank are gradually brought to rest. During the fall of the tank, and before it comes to rest, a chain attached to the end of the valve-stem in the tank is brought into tension and arrests the descent of the

Fig. 369.—Olsen Cement-testing Machine.

valve before its seat stops descending. The opening of this valve allows the contents of the tank to be quickly discharged into a hopper placed upon the floor, and is then carried off through a waste-pipe to the sewer. As soon as the tank has discharged its contents, the weight on the left end of the lever brings the lever and tank into the position shown in the illustration, the valve taking its seat during this movement, and the machine is ready for another break. The actual load can be applied at from 0 to 80 lbs. per minute, thus giving an increase of stress of from 0 to 1200 lbs.

per minute. The speed generally used is 400 lbs. per minute, and with the valve set for this speed the needle-beam will float every time within ⅕ second of the proper time.

"The stress on the specimen is measured by a poise travelling on a graduated scale-beam, which can be read by means of a vernier to 1 lb. and can

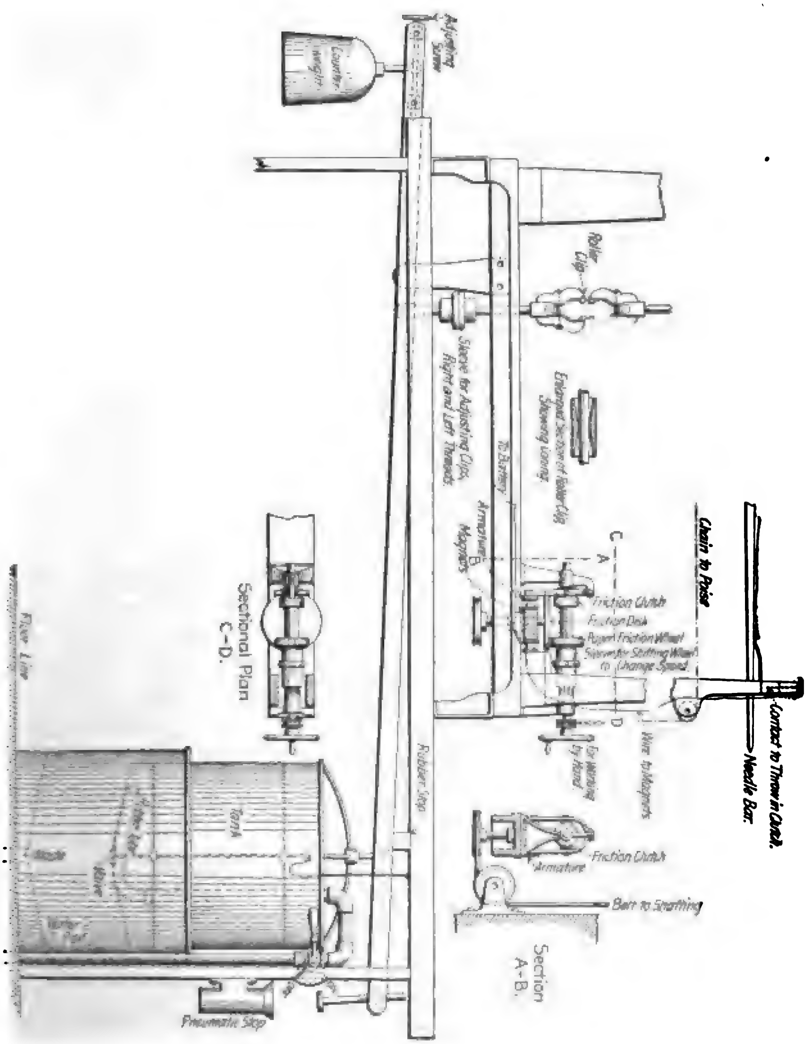

FIG. 370.—The Porter Cement-testing Machine.

be moved automatically or by hand at the wish of the operator. The automatic movement is accomplished by the following-described device:

"The horizontal disk and its engaged friction-wheel are driven continuously by the pulley placed at the lower end of the vertical shaft and belted to overhead shafting. This friction-wheel is feathered to a sleeve that runs

loose on its shaft and carries a coned clutch that is nominally disengaged from its cone, which is also feathered to the shaft, and can be moved slightly longitudinally on the shaft into contact with the clutch by the action of the vertical lever.

"When the needle-beam rises, it makes contact through a vertical pin in the top of the frame, which completes an electric circuit and sends a current through the electromagnet and causes it to attract its armature at the lower end of the vertical lever, which moving to the right engages the friction-clutch and causes the shaft to revolve. This shaft operates the sprocket-wheel and chain which draw out the poise on the scale-beam until the needle-beam drops, breaking the electric circuit. Breaking the electric circuit releases the armature and allows the friction-clutch to disengage, and the poise comes to rest. The friction-wheel may be set at a greater or less distance from the centre of the disk by turning the capstan-head nut, and the chain is overhauled faster or slower, causing the poise to move accordingly. If desired, the poise may be operated by the hand-wheel without interfering with the automatic device other than cutting out the circuit. The chain is attached to the poise in line with the three knife-edges of the scale-beam; hence the tension in the chain has no tendency to lift up or pull down the poise. This point is often overlooked in designing this detail, not only in cement machines but in testing-machines in general. The writer [Prof. Porter] has a cement machine in which the error due to this cause is over 15 lbs.

"This machine as described has been in almost constant use for eighteen months and has given entire satisfaction. The operator has simply to place the briquette in the clips, open the supply-valve, wait until the briquette breaks, and then note the reading on the scale-beam. The objection to this machine is the space it occupies, requiring a floor-area of 7 × 2 ft., and the necessity of a constant head of water."

325. Importance of an Exact Central Position in the Clips.—It was shown in Art. 26 that if h = width of specimen and a = eccentricity of loading, the percentage of increase in the stress from this cause is given by the fraction $\frac{6a}{h}$. Thus if a cement briquette 1 inch thick be placed in the clips 0.01 inch out of centre, its strength will be reduced by 6 per cent. This assumes perfect freedom of motion of the clips at the surfaces of contact, which they do not have. Experiments made at the Massachusetts Institute of Technology * have shown that a displacement of $\frac{1}{16}$ inch decreased the tensile strength by from 15 to 20 per cent (see Fig. 371).

326. Compression Tests of Cement have not been common in America, though long practised in Europe. The excellent relation indicated in Fig. 337, p. 419, between the tensile and the compressive strength of Portland-

* *Trans. Am. Soc. Mech. Engrs.*, vol. IX. p. 181.

cement mortar (1 C. to 3 S.) would seem to show that both tensile and compressive tests are not required, and American engineers have always acted on this assumption.

The French Commission recommend compression tests, however, in addition to the tension tests, but they do not advise the making of separate test-specimens. With the form of briquette shown in Figs. 355 and 356 the line of rupture is definitely fixed (with very few breaks outside the grooved section), and hence the two halves of the broken briquette will be nearly equal to each other and to all other broken parts. These ends are then to be tested

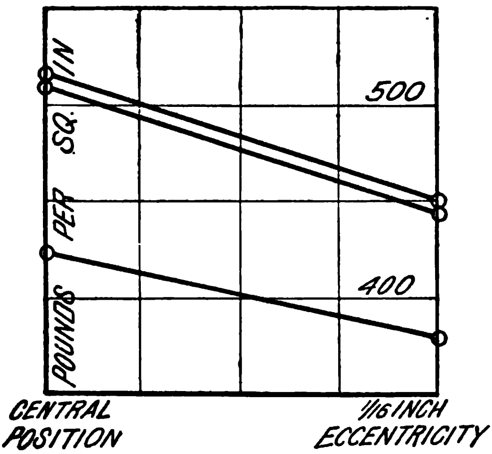

FIG. 371.—Showing Effect of Eccentric Position of Briquette in Clips. (*Assoc. Eng. Soc.*, vol. VII. p. 207).

by crushing, the force to be applied normally to its bed, and the sum of the test loads on the two ends of one briquette to be the crushing strength of that specimen. In the absence of such broken briquettes to serve for this test, cylinders of the same area and height are to be made up and tested.

Since this height is but 22 mm., while the diameter of the equal cylinder is 45 mm. (Fig. 355), the specimen has a height of but one half its lateral dimension, and hence the compressive strength of such a form of specimen is 20 per cent greater than that of a cubical form as shown in Art. 22, Fig. 17. The broken briquettes are chosen to avoid making up additional specimens, and also because this insures identical material for both the tension and the compression tests. For instituting a comparison with the compressive tests on other material, where the cubical form has been almost universally used, the correction coefficient of 0.83 can be employed, as stated above, or else cubical specimens can be prepared and tested.

It is of course necessary to prepare all compressive-test specimens with care, by reducing the two bearing-surfaces to true planes. It would also be wise to provide a universal joint back of one of the bearing-plates.*

* See method employed at the Massachusetts Institute of Technology, *Am. Soc. Mech. Engrs.*, vol. IX. p. 172.

In America compression tests of cement are made on the universal testing-machines so common in this country. In Europe many special machines are made for this purpose, one of the most recent of which is shown in Fig. 372. Here the load is indicated by the position of the radial arm moving

FIG. 372.—Machine for Making Tests of Cement in Compression. (Manufactured by Amsler-Laffon & Son, Schaffhausen, Switzerland.)

over the graduated arc, the actual movement of the upper head being thus multiplied a known number of times. As this movement is resisted by a powerful helical spring, when this spring has been standardized its compression is a true index of the load.

327. Cross-bending Tests of Cement have been advocated occasionally, but they have not come into general use anywhere. The French Commission

have also undertaken to standardize this test. They recommend a specimen 5 inches (120 mm) long and 0.8 inch (20 mm.) square in cross-section, and they show how this specimen may be broken on the Michaelis machine, Fig. 364, by attaching the centre-bearing, upward-pulling stirrup to the small hook at the left end of the lower lever.

M. Durand-Claye has shown by very extended series of tests in tension and in cross-bending, on identical samples of neat Portland cement, that the average ratio of the modulus of rupture in cross-bending to the tensile strength, as determined upon standard forms of briquettes, is 1.92 at 7 days and 1.86 for 28 days, or an average of 1.89.* This relation was found to subsist between averages made up from the means of the three tests in each set of six, in both tension and cross-bending. The mean error of a single test at 28 days was found to be 2.10 per cent for the tension tests and 2.13 per cent for the tests in cross-bending, thus showing that the two methods of testing were equally accordant.

It would seem, therefore, that tests in cross-bending may be employed with assurance as a means of determining both relative and absolute values of cements and cement-mortars, their principal disadvantage lying in the fact that there are few records extant with which to compare the results of such tests.

The principal recommendation for the use of transverse tests would seem to lie in the economy of a testing outfit. It has been estimated that a suitable machine for testing cement transversely could be constructed for about $12, while a set of moulds for sixteen prisms would cost not to exceed $3, or if these latter be made of cast iron the cost need not exceed $5 per set of twelve after the patterns are made.†

It is further claimed that since all transverse breaks are fair, while with the forms of briquettes and clips hitherto used in America nearly fifty per cent of the breaks occur outside of the minimum section, the results of transverse tests must be more reliable. If, however, a form of briquette and clip can be devised which will always give fair breaks, this claim of advantage will no longer stand. There seems to be now in this country no inclination to change from tension to transverse tests of cement.

328. Standard Tests to Determine the Adhesion of Cement-mortars to Various Substances.—While the tensile strength of briquettes shows the cohesion of the mortar, it has been found by experiment that its adhesion either to other mortars or to the same mixture which has already hardened, or to brick or stone or metal, is very much less than its cohesion. It is important, therefore, to have a standard test of adhesion, as well as of

* Messrs. Abbott and Morrison, in their thesis published in *Engineering News*, Dec. 14, 1893, show that for neat cement this ratio was 1.8 on prisms one inch square and broken on a span of four inches.

† See *Engineering News*, vol. xxx. p. 469, where complete detail drawings are given of both the machine and of the moulds.

strength. Because tests of this kind are comparatively new, no general custom has been established in America on the subject; but the following recommendations have been made by the French Commission:

(1) For tests of adhesion of cements and cement-mortars use will be made of a special form of briquette, moulded in two parts, these two parts consisting of the two materials whose adhesion is to be tested, provided both can be moulded, or containing between them a prism of the solid body to which the adhesion of the mortar is to be determined The form of this briquette, as modified for English units, with one square inch of area on the surface of adhesion, is shown in Fig. 373. This mould is formed in two parts, and is used to form in succession the two halves of the complete briquette.

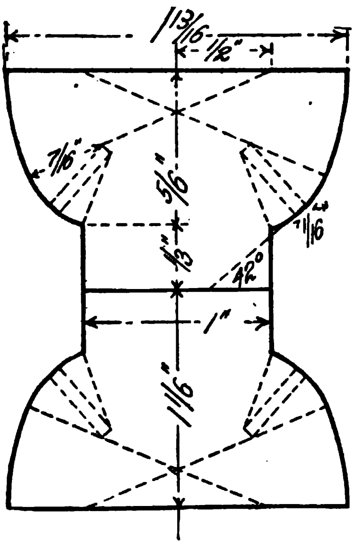

FIG. 373.—Form of Briquette for Adhesion Test of Cement as adopted by the French Commission and adapted to English Units.

(2) *To compare the force of adhesion of different cements to a given material, normal adhesion-blocks* will be prepared as follows: Use for these always one kind of standard Portland cement which has passed a sieve of eighty meshes to the linear inch, mixed with the standard sand No. 3 (see Art. 317) in the proportion of one of cement to two of sand. These normal adhesion-blocks will be moulded in the form of one half of the briquette shown in Fig. 373.* It will be gauged with 9% of water and rammed into the mould. At the end of 24 hours in air it will be placed in fresh water for a period of at least twenty-eight days. When it is to be used, it will first be dried and its adhesion-surface polished with emery-paper.

(3) The cement to be tested for adhesion with these standard blocks prepared as above will be mixed as a normal plastic mortar, one of cement to three of sand (see Art. 316), which will be introduced into the mould with a trowel, this mould now being placed with a normal adhesion-block at the bottom in place of the movable metallic disk. The mould will remain upon this completed block until it is ready for testing, and the block will be allowed to harden either in air or water, and for such period as the test requires. It is recommended that the number of tests, the periods of time, the methods of hardening, and the recording of the results should comply with the conditions given for tension tests in Art. 319.

* The detail drawings of these moulds are given in the *Fr. Com. Rep.*, vol. IV. p. 284.

(4) *To compare the force of adhesion of a given cement to different materials.* For this purpose the test-specimens will be prepared as described above, except that in place of the normal adhesion-blocks similar blocks of the various materials to be tested will be prepared and allowed to harden, provided these are such as can be moulded in this manner. If such materials are solid, small disks, about three-eighths of an inch thick, will be prepared, and these will be used in the bottom of the mould in place of the metallic disk, the adhesion-block to be completed by using neat Portland-cement mortar. After this has hardened the briquette will be completed by making the other half of a standard plastic mortar, one cement to three of sand, using the particular kind of cement whose adhesion to these various substances is to be tested.

If the normal plastic mortar is not used in adhesion tests, a full description of its composition should be indicated on the records.

These adhesion-briquettes to be broken on a standard tension-testing machine, using the regular tension-clips.

329. Normal Variations in Volume of Cement-mortars in Air and in Water.—From elaborate tests on the swelling and shrinking of cement-mortars hardening in air and under water made at the Massachusetts Institute of Technology, Boston, and by Professor Bauschinger at Munich, it may be stated:

1. Cement-mortar hardening in air shrinks almost uniformly for a period of more than three months, the linear shrinkage in that time being, for neat cement, from 0.12 to 0.34 of one per cent, and for cement-mortar, one of cement to one of sand, from 0.08 to 0.17 of one per cent. The change in volume is of course three times the above percentages.

2. Cement-mortars hardening under water increase in linear dimensions from 0.04 to 0.25 of one per cent in three months for neat cement, and from 0.00 to 0.08 of one per cent for a mortar composed of one part cement to one of sand; the volumetric expansion being three times these amounts.

Professor Bauschinger found for nine German Portland cements, neat, an expansion, when hardened under water, of 0.05 of one per cent in sixteen weeks; Mr. Grant found for English Portland cement an expansion of 0.08 of one per cent in three months, this latter figure agreeing with the average results of the tests made on eight kinds of Portland cement at the Massachusetts Institute.*

For mortars composed of one part of cement to three of sand the variations in volume are very much less than those given for mortars of equal parts of sand and cement.

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* See progress reports of committee of the Am. Soc. C. E. on the compressive strength of cements, vol. XVII. p. 215, and vol. XVIII. p. 264.

330. Recommendations of the French Commission for Testing Permanency of Volume.—Tests of permanency of volume may be of two general classes—*cold* and *hot*.

Cold Tests will be made upon thin cakes of neat-cement paste made up on glass, about six inches in diameter and ½ inch thick at the centre, with thin edges, and placed immediately in water or in air, along with the briquettes which are hardening in these two media. These cakes to be examined at periods of 7 days, 28 days, 3 months, 6 months, 1 year, 2 years, etc., corresponding to the like periods for the tests of strength.

To measure the amount of the linear change of volume of neat cement immersed in cold water, a small cement form, 32 inches long and ½ inch square in section, may be moulded and placed vertically in a glass tube 1 inch in diameter filled with water. The expansion would be indicated by the movement of the long arm of a lever over a graduated scale, which is actuated by a pin embedded in the upper end of the specimen when made.*

Evidently it may require years to assure one of the permanency of volume, or soundness, of a cement by the use of the cold-water and air test.

Hot Tests to be made on cylinders of neat cement 1¼ inches (30 mm.) high and 1¼ inches in diameter, made up and left in metal moulds composed of sheet metal 0.02 inch (0.5 mm.) thick (No. 25 gauge). This mould to be entirely severed on one

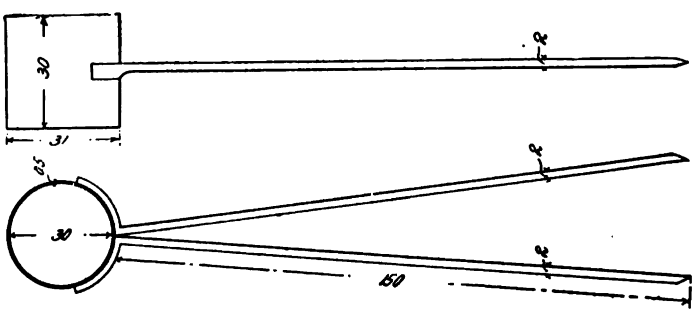

FIG. 374.—Apparatus for Testing Permanency of Volume of Cement. (Recommended by the French Commission.)

element, and to have soldered to it on the opposite side two arms, six inches long, forming an angle with each other as shown in Fig. 374. The decreasing distance between the extremities of these arms to be a measure of the swelling of the cement.

These moulds to be immersed in cold water as soon as filled, and allowed to set for 24 hours, or for a shorter period if it is a quick-setting cement. The mould will then be placed on a grating in a vessel of water and its temperature raised to the boiling-point in from 15 to 30 minutes. This temperature is to be maintained for six hours, when the water will be allowed to cool down before removing the specimen for remeasuring the distance between the six-inch arms.

This hot test not to be applied to natural cements, or to any cement which sets very rapidly.

The consistency of the cement used in both the hot and the cold tests to be of the *normal consistency* described in Art. 316.

331. The Permeability of Cement-mortar is often a very important matter, as in the case of reservoir wells and linings, and often in foundation-walls placed below the level of the ground-water. Neat cement-mortar is absolutely impervious when it has hardened and has not cracked, and so also is a mixture of one to one, or even of two of sand to one of

* See Fig. 25, p. 302, vol. I, Report of the French Commission, 1895.

cement, by weight, if well mixed. The normal mixture of three of sand to one of cement may also be made practically impervious with the most thorough mixing of the dry ingredients and a compacting of the mortar by hard ramming.

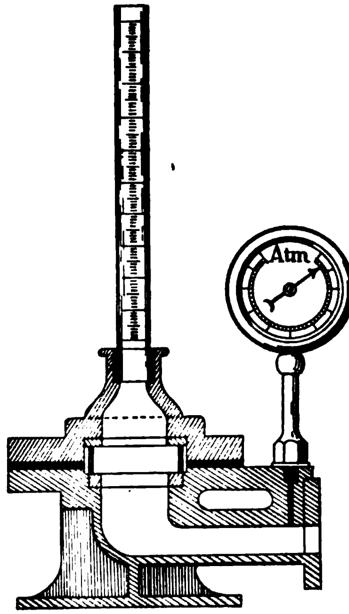

FIG. 375.—Tetmajer's Apparatus for Testing the Permeability of Cement-mortar. (*Communications*, vol. VI.)

Professor Tetmajer has used the apparatus shown in Fig. 375 to obtain a modulus of permeability. Here a cylinder of the mortar is made and allowed to harden under water for a specified time. It is then mounted in the apparatus by means of annular rubber-cushion or packing disks, and the water let on below under a known pressure. The permeability of the mortar is indicated by the rate at which the water passes the disk and rises in the glass tube above, which is graduated to cubic centimeters. The author has also used this apparatus with satisfactory results, a convenient pressure to use being that of the city water-mains.

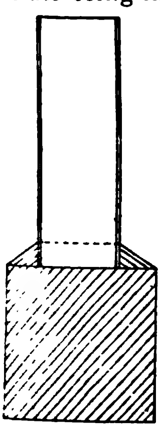

FIG. 376.—Apparatus for Testing the Permeability of Cement-mortar. (Recommended by the French Commission.)

The French Commission recommend a standard permeability test as follows:

(1) The permeability of cement-mortars will be indicated by the number of liters of water passing per hour through a cubical block of 7 cm. (say 2¾ inches) on a side, under the following conditions.

The water will be brought to the top face of the specimen, laid edgewise (what was the horizontal plane in the formation of the cube now becoming a vertical plane), through a glass tube, 35 mm. internal diameter and about 4 or 5 inches high, which is sealed to the top face of the cube by neat cement-mortar as shown in Fig. 376. A rubber tube connects the upper end of the glass tube with the reservoir placed at a height (from the surface of the water of immersion to the surface of the water in the reservoir) of 4 inches, 40 inches, or 400 inches (0.1 m., 1.0 m., or 10.0 m.).

Before beginning the experiment the cube of mortar to be immersed in water for 48 hours, and during the test the block is to remain immersed to prevent the formation of an impervious coating on the outside from the evaporation of the exuding water.

The volume of water passing will be given for the standard periods of

24 hours, 7 days, 28 days, and 3 months. For very porous mortar a shorter period than 24 hours may be employed, and at the same time the head of water used must be stated.

Tests will be made on three similar specimens, the mean of the two most accordant results to be used.

(2) The *normal test* of permeability will be made on cubes made up of *normal plastic mortar* (3 sand to 1 cement, by weight) as described in Art. 319, and the specimen cubes must harden in water under the normal conditions for 28 days before testing.

For tests on other mixtures, and for other periods of hardening, they recommend that mixtures of 2 sand to 1 cement, and 5 sand to 1 cement, by weight, and hardening periods of 7 days, 28 days, and 3 months be chosen.

In all cases the composition, age, and conditions of hardening must be stated, as well as the amount of water passed and the pressure-head used.

332. Tests for the Decomposing Action of Sea-water.—As a result of the porosity of cement-mortars and concretes and of the resulting action of sea-water on the interior of the mass, producing therein certain changes partly by solution and partly by the formation of new chemical compounds, cement-mortars and concretes are often disintegrated when subjected to the action of sea-water. The French Commission have carefully studied this question, and have recommended the following test, which they regard as of value in determining the comparative resistance of cement to this action:

(1) Standard tension briquettes of normal plastic mortar (one cement to three of sand) will be made, and after 24 hours in air will be placed in sea-water which is to be renewed every two days during the first week, and every week thereafter. During the first week the volume of this sea-water to be at least four times that of the briquettes immersed in it.

An equal number of duplicate briquettes to be exposed in a similar manner to the action of fresh water. Tension tests on these duplicate briquettes to be made at the standard periods of 28 days, 3 months, 6 months, 1 year, etc., and the effect of the sea-water to be shown by a comparison of the results.

(2) Filtration tests will be made on specimens having a cubical form, as described in Art. 331, and to be exposed to the action of sea-water both in the bath and in the filtration reservoir there described. The head will be 4 inches, 40 inches, or 400 inches, according to the permeability of the specimen. Two sets of duplicate test-specimens will be subjected to this test, one having hardened in sea-water and the other having hardened in air for the several standard periods chosen.

A third duplicate set of exactly similar cubical blocks to be preserved and hardened in fresh water for the same periods of time.

In the absence of actual sea-water, artificial sea-water will be prepared with the following formula:

Chloride of sodium ($NaCl$)................................	30 g.
Sulphate of magnesia, crystallized ($MgOSO_3,7H_2O$)...........	5
Chloride of magnesium, crystallized ($MgCl,6H_2O$)...........	6
Sulphate of lime, hydrated ($CaOSO_3,2H_2O$)................	1.5
Bicarbonate of potassium ($KOH_2O,2CO_2$)................	0.2
Distilled water..	1000

All the above cubes to be superficially examined and tested in compression at the standard periods chosen.

The following observations will be taken:

(*a*) The comparative appearance of the specimen subjected to the several kinds of treatment.

(*b*) The tensile strength of the two sets of briquettes which had hardened in salt and in fresh water.

(*c*) The compressive strength of the three sets of cubical blocks which had hardened in salt water and in air, and which had been subjected to the filtration tests, and the blocks which had hardened in fresh water.

(*d*) Chemical composition of the cubical blocks subjected to the several kinds of treatment.

For other compositions, mortars composed of one cement to two of sand, and one cement to five of sand, to be chosen and tested at the standard periods of 7 days, 28 days, 3 months, etc.

CHAPTER XXII.

TESTS OF THE STRENGTH OF STONE AND BRICK.

TESTS OF STONE.

333. Tests of the Strength of Stone Limited to the Crushing Test.—Since stone can readily be prepared for crushing tests, these have been almost exclusively employed in determining its strength. In view of the data obtained from comparative tests of cement in tension and compression shown in Fig. 337, p. 419, it might be inferred that the crushing test would show also the relative strength in tension. Since failure in crushing is a failure by shearing, it might be supposed that the true relative shearing strength would also be shown by the compression test. When stone fails in cross-bending it breaks first on the tension side of the beam, and hence this is a failure in tension, and therefore the crushing test has been thought to give correct relative values of cross-breaking strength.

These assumptions prove not to be correct, however, as has been very conclusively shown by Bauschinger in volumes X, XVII, and XIX of his Communications, where the results of tests on more than a thousand specimens of building-stones of the various kinds found in Bavaria are given. These tests were made in compression, in tension, in cross-bending. and in shearing, and no fixed relation can be given to these several kinds of strength. Probably this is largely due to the fact that stones are not amorphous bodies, but are usually either sedimentary or crystalline or both, with definite planes of cleavage and of weakness. Since stone is used, however, almost exclusively in compression, it is usually considered sufficient to test its strength in compression only.

The conditions to be fulfilled in the crushing test of stone are sufficiently elucidated in Chapters III and XVI. While the test-specimens should have heights greater than their least lateral dimensions, yet in order to make the results comparable with those hitherto obtained and recorded it is necessary to continue to make these tests on cubical forms.

TESTS OF PAVING-BRICK.

334. Kinds of Tests Required.—The use of brick for the wearing surface of street-pavements is now so universal that this new product, "vitri-

fied paving-brick," has become one of the most important of the materials employed by the civil engineer. Since appearances in this material are entirely untrustworthy, and since these products vary greatly not only as between the output of different manufacturers, but also as between different kilns of the same factory or even in different parts of the same kiln, a thorough system of mechanical tests to determine the probable wearing qualities is absolutely essential. To develop these qualities four tests are now commonly accepted as essential, namely *:

1. Cross-breaking.
2. Crushing.
3. Impact (the rattler test).
4. Absorption.

335. The Cross-breaking Test.—This is made on single whole bricks by setting them edgewise on two rounded knife-edge bearings about 7 inches apart and loading them at the centre. In order to insure a true bearing of the knife-edges the brick should be ground to true parallel surfaces, or else the lower bearings should be rounded longitudinally sufficiently to prevent a twisting or torsional action. The cross-breaking modulus of rupture is found by applying the formula

$$f = \frac{3Wl}{2bh^2}.$$

336. The Crushing Test is usually made on a half-brick, set edgewise, and one or both of the ends of the brick previously used in the cross-breaking test may be used. As these faces are very rough in paving-brick (from their having been reduced to a semi-plastic condition in the kilns, these being the bearing-surfaces), it is impossible to make fair crushing tests on these forms without grinding them to true parallel planes. This can readily be done on a regular stone- or marble-grinding table operated by steam-power, such being available in all large cities. When this is done they may be bedded on single thicknesses of tar-board; or placed directly between steel plates in the testing-machine. Great care must be exercised to place the specimen centrally in the machine, and to see that the bearing-plates fit evenly upon the specimen. One of these plates should have a spherical base to make it adjustable and so insure an even and true bearing on the test-specimen. The specimen should fail all at once, with a loud report, with little or no previous spalling. The load must be increased very slowly and uniformly, and the weighing-beam automatically balanced if practicable.

337. The Rattler Test.—Formerly this test was made as an *abrasion* test by using a great quantity of small castings. The author has long insisted that this test should partake of the character of an *impact* test, and this view now generally prevails. Paving-brick are broken to pieces in service rather than worn or ground down, and the property of resilience is the one sought rather than hardness. The standardizing of this test

* See Recommendations of a Committee of the National Association of Brick Manufacturers, in Art. 338.

has proved a difficult task, but it has been fairly accomplished by Mr. F. F. Harrington, a former student of the author's, now in charge of the Testing Laboratory of the Board of Public Improvements of the City of St. Louis. Mr. Harrington's rattler is a cast-iron barrel, polygonal in form and having fifteen staves, similar to that shown in Fig. 377. Its length is 42 inches and its diameter is 24 inches, and it revolves on trunnions at

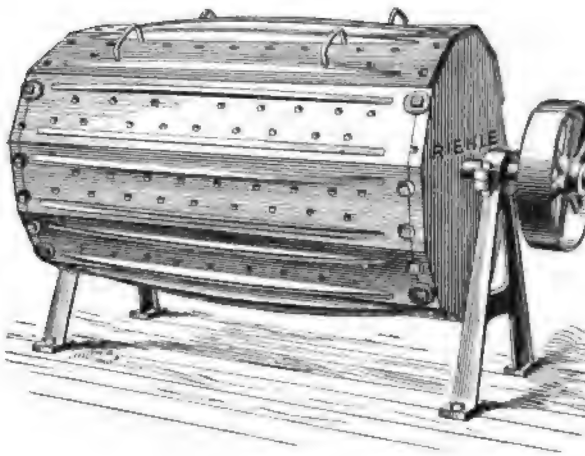

FIG. 377.—Rattler for Testing Paving-brick.

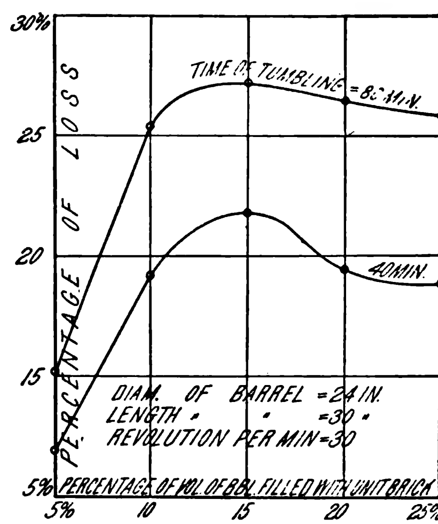

FIG. 378.—Rattler Test of Brick showing Maximum Impact-effect when the barrel has 15% of its volume filled with brick. (Harrington.)

the ends. A movable cast-iron partition can be inserted on the inside so as to shorten the length of the part used to any amount less than 42 inches.

It is operated by an electric motor which is also used for other purposes in the laboratory.

Fig. 378 shows the results of placing different amounts of brick in the barrel. Evidently there would be a particular amount (percentage of volume) which would give a maximum impact-effect. This proves to be 15%. That is to say, when 15% of the volume of the interior of the barrel is filled with brick, solid measure,* the impact-effect was a maximum, the barrel making 30 revolutions per minute. This quantity was then adopted as the quantity of material always to use.

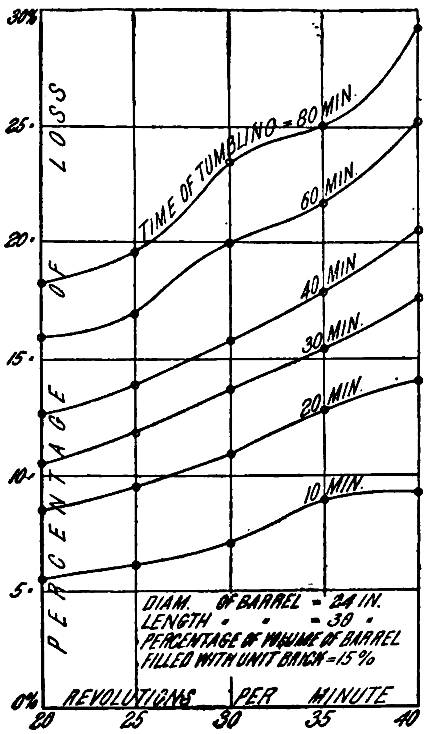

FIG. 379.—Showing the Effects of Time in the Rattler Test of Paving-brick (Harrington.)

In Figs. 379 and 380 the effects of time and speed are shown when the standard quantity of brick (15%) was in the barrel. Since the 60-minute curve gave an even 20% less at 30 revolutions per minute, this being regarded as about the proper amount, this product, of 1800 revolutions, was chosen.

The effect of the length of the barrel is shown in Fig. 381, all being filled to 15% of the total volume. It will be seen that the length of the

* This means that the total solid contents of the brick equals 15% of the volume of the barrel.

barrel has no sensible influence on the tests, provided it is always filled to the same percentage of its volume.

338. Standard Tests of Paving-brick.—As a result of these experimental tests, and of similar ones carried out by Prof. Edward Orton, Jr., of the Ohio State University, chairman, a committee of the National Association of Brick Manufacturers of America, appointed in Feb. 1896 reported in Feb. 1897, recommending the following tests as standard:*

1. A rattler test, made in a cast-iron rattler 28 inches in diameter and 20 inches long, having fourteen flat sides with one-fourth-inch spaces intervening. The rattler to be filled with a number of any given kind of brick equalling in total volume 15% of the volume of the rattler (requiring 1800 cu. in. of brick volume, or from 20 to 24 brick for this standard size). The rattler to be run 1800 revolutions at the rate of 30 revolutions per minute. In no case must a different kind of brick or other material be used to make up the charge. Other sizes of rattler, from 26 inches to 30 inches diameter and other lengths, could be allowed, and the speed might vary between 24 and 36 revolutions per minute.

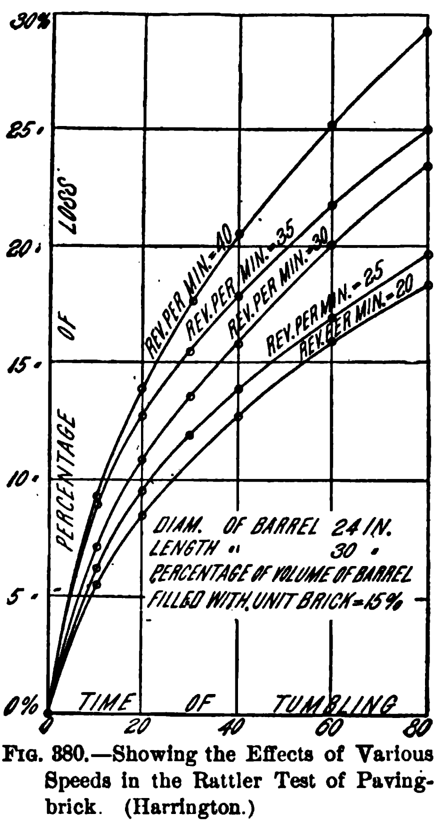

FIG. 380.—Showing the Effects of Various Speeds in the Rattler Test of Paving-brick. (Harrington.)

Two such tests on any given species of brick to constitute a *standard rattler test*, and the average result to constitute the record. This result to be a given percentage of loss of weight in terms of the original weight. The individual bricks do not need to be identified in the two weighings in this test.

2. A cross-breaking test (number of tests not stated) to be made as described in Art. 335, the lower knife-edges to be rounded to radii of 15 inches longitudinally and $\frac{1}{8}$ inch transversely. The span to be 6 inches.

3. A crushing test as described in Art. 336, or with the undressed bearing surfaces embedded in plaster of paris.

Tests 2 and 3, for strength, were not regarded by the committee as essential, but were made optional

* The author, on invitation, participated in the proceedings of this committee.

The absorption-test was condemned as misleading, inasmuch as no brick which could endure the proposed rattler test would ever absorb enough water to injure it; while if the test be used, a thoroughly vitrified (glassy) brick might be preferred to a semi-vitrified one because of its smaller absorption.

It will thus be observed that the committee regarded the rattler test, as here proposed, as quite sufficient to determine the wearing and weathering qualities of paving-bricks. This test alone requires from 40 to 50 brick to be furnished, since it is to be made in duplicate. As a matter of conven-

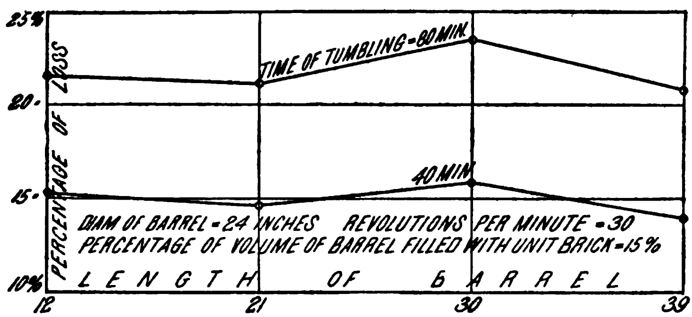

FIG. 381.—Showing Effect of Length of Barrel in the Rattler Test of Paving-brick. (Harrington.)

ience, therefore, the rattler should be made double, or so as to contain two apartments having the dimensions described above.*

The first effect observed in the rattler test is one of chipping on all edges. After these have rounded off somewhat the effect is more evenly distributed over the outer surface, but it still remains principally an impact action. The dust and small pieces fall through the spaces left between staves for this purpose, so that the rattler remains comparatively clear of débris. If absorption-tests are made, they should be made on bricks which have passed the rattler test, because their glazed surfaces are then broken, if not largely removed.

Accordant results could not be obtained when different kinds of brick were put in the rattler at one time or when other materials, such as stone, granite, or cast-iron blocks, were employed; hence the requirement that the full complement of material in the rattler should consist of the brick tested.

No specified requirements were named by the committee, as sufficient data had not yet been obtained.

* Standard drawings and patterns for such a rattler have been prepared by Mr. M. L. Holman, Water Commissioner of St. Louis, from which the author has had a machine constructed.

CHAPTER XXIII.

TESTS OF THE STRENGTH OF TIMBER.

339. The Variable Strength of Timber.—As shown in Chapter XIII, sound timber of a given species varies in its strength from two general causes, its structure and its moisture condition. Neither of these sources of strength (or weakness) has hitherto received proper study and analysis, and hence the known variations in the strength of timber has been attributed either to its inherent and undiscoverable variations, or to variations in the size of the sticks tested. Some of the most important conclusions to be drawn from the U. S. Timber Tests are:

1. *The strength of timber is about twice as great when it is dry as when it is green or wet.**

2. *The strength of a given species of timber at a given percentage of moisture is governed by the ratio of the summer (solid) to the spring (open) wood, or in other words by its specific gravity or solidity.*

3. *The strength per square inch of a large stick in every kind of test is fully equal to that of a smaller stick cut from it when both are similarly proportioned and similarly free from faults.*

4. *It is very highly probable that the strength of all kinds of wood-fibre of like structural arrangement (thus putting the oaks into a separate class) increases directly with the specific gravity (or weight per cubic foot) of the dry wood.* (See a discussion of this subject in Chapter XXXII.)

The moisture state is the great and governing cause of variation in strength. When reduced to the same moisture condition it may be said, as a result of about 40,000 tests of timber made by the author, that in crushing endwise 90 per cent of all tests fall within 25 per cent of the mean, and 55 per cent of all tests fall within 10 per cent of the mean-value for that species, and this is about as much as can be said for other kinds of building materials when all have been subjected to a reasonable inspection.

340. "The United States Timber Tests," † so called, were inaugurated in 1891 by Dr. B. E. Fernow, Chief of the Forestry Division of the U. S. Agricultural Department, and have been carried on with frequent interruptions ever since. They consist of a very complete series of investiga-

* See the curve showing variation of strength with moisture in Chapter XXXII.

† See Bulletins 6, 8, 10, 13, and others to be issued from time to time by the U. S. Agricultural Department, Forestry Division, and to be had on application to the chief of that division.

tions into the habitat, conditions, and laws of growth, structure, strength, and other properties, seasoning, preservation, and decay, and finally the artificial cultivation of the useful timbers of the United States. The great importance of the timber industry (being second only to that of agriculture in this country), and the universal absence of accurate (scientific) knowledge on these subjects, have seemed to warrant the undertaking and prosecution of this the greatest series of physical investigations ever carried out. The field-studies and the collection of the material have been done by Dr. Charles Mohr; the structural investigations have been made by Mr. Filibert Roth in Washington; and the mechanical tests have been made under the direction of the author in his testing laboratory at Washington University, St. Louis, Mo. The species examined and tested to date (December, 1896) are given in tabular form in Chapter XXXII. It there appears that there have been selected for these tests—

1. Sixty-eight trees of Long-leaf Pine* from South Carolina, Alabama, Mississippi, Louisiana, and Texas.

2. Twelve trees of Cuban Pine from South Carolina, Georgia, and Alabama.

3. Twenty-two trees of Short-leaf Pine from Alabama, Missouri, Arkansas, and Texas.

4. Thirty-two trees of Loblolly Pine from South Carolina, Georgia, Alabama, and Arkansas.

5. Seventeen trees of White Pine from Michigan and Wisconsin.

6. Eight trees of Red (Norway) Pine from Michigan and Wisconsin.

7. Four trees of Spruce Pine from Alabama.

8. Twenty trees of Bald Cypress from South Carolina, Mississippi, and Louisiana.

9. Four trees of White Cedar from Mississippi.

10. The test specimens of Douglas Spruce were not taken from selected trees, but were obtained from lumber shipments to the St. Louis markets.

11–20. Eighty-three trees of ten species of Oak from Alabama, Mississippi, and Arkansas.

21–27. Twenty-four trees of seven species of Hickory from Mississippi.

28–29 Five trees of two species of Elm from Mississippi and Arkansas.

30–31. Four trees of two species of Ash from Mississippi.

32. Seven trees of Sweet Gum from Mississippi and Arkansas.

Besides these a great many small trees were taken, from which disks were cut at frequent intervals, from butt to top, and sent to Washington for the physical and structural studies Complete field notes were taken, also, of the geographical position, the immediate surroundings as to forest growth, character of soil, moisture conditions, etc. The diameter of the stump, the age of the tree, and the distance to the first limb were also noted.

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* Sixteen of these were "bled" trees to determine the effects of "boxing."

All this material has come from the Southern States except the white and Norway pines. The trees have been selected and cut by Dr. Mohr; the logs cut from them were shipped to the author at St. Louis in car-load lots. Disks 8 inches long have been taken from all these trees, at a number of heights, and also from many trees of the same species too small for timber-test specimens, and sent to Mr. Roth at Washington. The logs have been cut into test-timbers in various ways as shown in Fig. 352. The largest forms are full-sized beams (from the 18-foot logs) and columns (from the 12-foot logs), the size depending on the size of the log. The intermediate forms are 4 inches square, the standard size of the "small" sticks. The

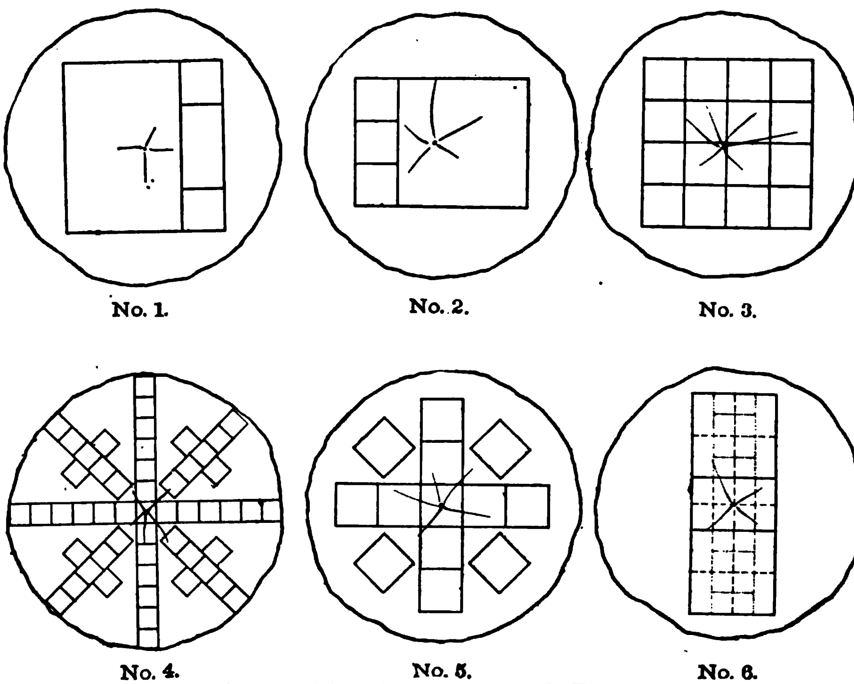

FIG. 382.—Showing Methods of Sawing U. S. Timber Test Logs.

smallest size is 2 inches square, which has been employed only in "special investigations." The form in No. 6 indicates that two or three large sticks are to be cut and tested to failure, from the uninjured portions of which are afterwards cut smaller sticks which are also tested for the purpose of comparing the strength of large and small sizes. Form No. 5 has been adopted as the standard method of cutting when only 4-inch sticks are taken. Only logs over 24 inches in diameter at the small end could furnish this entire system of sticks, smaller logs giving the five interior ones only. The logs are always laid out on their upper ends, taking the pith as the centre of the diagram regardless of how unsymmetrical this may

lie in the cross-section of the log. The logs are always 12 or 18 feet in length, and the 4-inch test-sticks are cut to 6-foot lengths, thus getting two or three such lengths from each 4-inch stick shown in the log diagram.

341. The Mechanical Tests.—As a rule the following tests have been applied to every 4-inch stick:

1. Cross-bending.
2. Crushing endwise of the grain.
3. Crushing across the grain.
4. Shearing along the grain.
5. Tension.

Since June 1895 no tests have been made in tension, as it was thought this kind of strength was so great as to remove it from the category of possible methods of failure. It was thought timber would never fail in pure tension in practice.

For each and every test a section of the stick about $\frac{3}{16}$ inch thick is cut from near the point of failure, and used for determining the percentage of moisture as described in Art. 343.

The test-sticks were subject to a system of inspection and rejection which it was thought would correspond to such a system in actual practice where the timber was to be used in structures where the parts are proportioned to their loads, and hence it is thought the average of all the results fairly corresponds to such an average strength in practice at similar stages of dryness.

In order to make the results comparable it was of course necessary to reduce them all to equivalent values at a standard percentage of moisture. For all reductions made previous to May 1896 this standard had been 15 per cent moisture, computed on the dry weight. After that date 12 per cent was chosen as better representing the condition of the roughly seasoned timber, whether in or out of doors. In a dry, heated building the moisture falls as low as 8 or 10 per cent.

342. The Cross-bending Test on the 4-inch sticks is made on a small 8000-lb. testing-machine designed by the author, shown in Fig. 301, page 370, while the large beams are tested on the 100,000-lb. machine shown in Fig. 302, page 371. In both cases the loads are applied so as to produce a uniform rate of deflection (with the 4-inch sticks it is always at the rate of $\frac{1}{3}$ inch per minute, while with the larger sticks it is $\frac{1}{4}$ inch per minute) in order to eliminate the time-effect, which is very large with timber especially under the higher loads.

With the small sticks two central bearing-points are used, 12 inches apart, thus putting that length of stick under the maximum bending-stress, and so really testing 12 inches in length of the stick instead of about 1 or 2 inches with a single bearing.* Of course in all cases the bearings are

* This was done as the effect of a paper by Prof. J. Burkett Webb before the Am. Assoc. Adv. Science, Section D, at Rochester, N. Y., 1892.

spread over a considerable area by means of steel plates to prevent the destruction of the fibres by crushing across the grain. With the large beams, an oak saddle some 30 inches long, of the full width of the beam, and rounded slightly on the bottom in a longitudinal direction, is used for the centre bearing under the knife-edge of the machine.

The deflections of the small beams are measured by means of a micrometer-screw bearing on the head of the power-screw, as shown in Fig. 301, while in the case of the large beams a thread was stretched (by a rubber band) along one or both sides from nails in the neutral plane above the end bearings, and readings taken on a scale tacked to the beam at the centre. The scale was nickel-plated and kept polished to act as a mirror, and the parallax of the thread on the scale was obviated by bringing the thread and its image into coincidence when the readings were taken. The readings were made to 0.001 inch with the small beams and to 0.01 inch with the large beams.

The formulæ of reduction for strength, modulus of elasticity, and resilience were adapted to the particular method of test employed; but since the resilience in inch-pounds per cubic inch is different for the two cases (single and double bearings at centre), the larger results obtained with a double bearing have been reduced to their equivalent for a single bearing to make them all comparable with each other and with the results usually obtained, which would be with a single bearing. The resilience of a beam loaded at the centre, in inch-pounds per cubic inch, is from eq. 6, p. 84.

$$r = \frac{1}{18}\frac{f^2}{E}$$

This is also the measure of the resilience of the outer ends of the beams tested with double bearings at the centre, while for the part between the two centre bearings, where the bending moment is uniform, it is, from eq. 10, p. 85.

$$r = \frac{1}{12}\frac{f^2}{E}.$$

The f being the same in the two cases, namely, the "apparent elastic limit" of the material, found by fixing a point of the bending-stress diagram where the rate of deformation is 50% greater than at the origin, as explained in Art. 13, p. 18.

The results obtained from each cross-bending test of timber, therefore, are:

1. Modulus of strength at the "apparent elastic limit."
2. Modulus of strength at rupture.
3. Modulus of elasticity.
4. Modulus of resilience, or springiness.

When large beams have been tested to failure, two smaller (4-inch) beams 6 feet long have been cut from the upper side of the larger beam at one end,

and two from the lower side at the other end, and these four small beams have been subjected to the same test to discover whether or not the ordinary formulæ are correct, or, what is the same thing, to discover whether the same values of the moduli named above would be obtained from both sizes. All the tests which have been made of this kind go to show that when both sizes are equally free from faults this is true. This proves that the strength of large sizes may safely be computed from tests on smaller sizes, *other things being equal.*

343. The Crushing-endwise Test.—For this test a section about 8 inches long of the uninjured portion of a 4-inch beam which had been tested in cross-bending, is taken by means of a circular cutting-off saw, and tested to failure in compression endwise. The stress-diagrams in this test are fairly indicated in Fig. 383. Failure occurs by a buckling down of the fibres as shown at *B*, Fig. 113, page 241. After this buckling action of the fibres across the entire section has occurred the strength of the specimen is only about 0.8 what it had been originally, as is shown in Figs. 29 and 383,

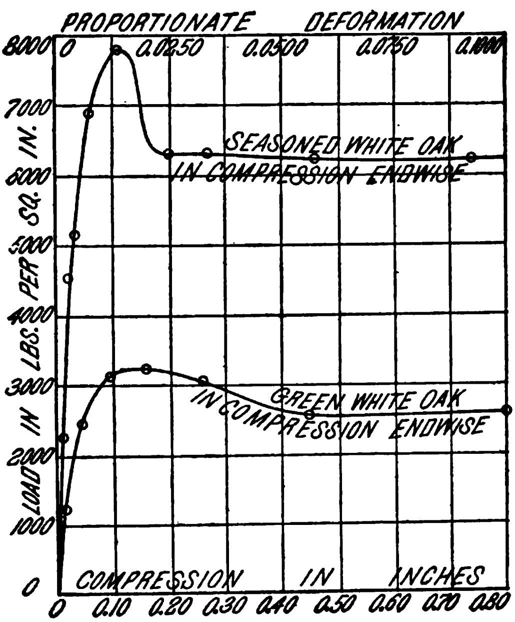

FIG. 383.—Typical Stress-diagrams of Timber when subjected to Compression Endwise.

this residual strength remaining about constant for largely increasing deformations.

This is the most valuable and characteristic single test to which timber can be subjected. It is the only one in which a relatively large stick can be evenly and simultaneously tested to failure throughout its entire cross-section. It should be expected, therefore, to give more uniform results than

any other, and such proves to be the case. It is also the simplest and easiest to make. For commercial purposes, therefore, this test alone would serve nearly every purpose, all the other various kinds of strength and stiffness being inferred from this one test.

344. Crushing Across the Grain.—Since timber is very weak in crushing across the grain, as compared to crushing endwise, this is found to be one of the most common methods of failure in practice. It is common to rest a timber column on a sill of the same wood, and to design the column for its maximum working load, paying no attention to the utter inability of the sill to carry this load without crushing. Many failures of timber structures are due to this cause alone.

As there is no definite point of failure in crushing across the grain, two limits of deformation have been arbitrarily chosen at which the load has been recorded, namely, at *three per cent compression*, as a working limit allowable, and at *fifteen per cent compression*, as an extreme limit, or as failure. The apparatus used to indicate these two limits, for heights (thicknesses) of specimen from 2 inches to 4 inches. is shown in Fig. 292 and explained in the text of Art. 282, p. 357. With such timber as oak, which has large medullary or pith rays, the crushing strength in a radial direction is greater than in a tangential direction.

345. The Shearing Test.—This is intended to develop the strength of timber to resist shearing along the grain. This strength is very small in nearly all kinds of wood, and may be reduced almost to zero by seasoning checks. It is a very common method of failure in timber framework, and hence it is important to test for it. The apparatus used is illustrated and described in Art. 300, p. 386.

346. The Tension Test.—The tensile strength of timber is so great (often over 30,000 lbs. per square inch) that it is difficult to make a fair test of timber in this way. Simple shouldering is out of the question, since the specimen shears out or the shoulders crush down. The author, after trying various methods, adopted the simple forms of specimens shown in Fig. 113, p. 241. This figure does not show the reduction of the cross-section at the centre, which was done by cutting out two segments of circles on the two sides by a band-saw, these segments having about an 18-inch radius. The reduced section left at the centre of the specimen was about 3 inches by $\frac{3}{8}$ inch, making something over a square inch of net section. These specimens were then gripped by flat, grooved, cast-iron wedges, in the 100,000-lb. universal testing-machine, and pulled to failure. Of course nearly straight-grained timber must be used or the failure is partly or wholly one of shearing. This test was finally abandoned altogether, as it was assumed that timber would never fail in practice in this way.

PART IV.

THE MECHANICAL PROPERTIES OF THE MATERIALS OF CONSTRUCTION AS REVEALED BY ACTUAL TESTS.

CHAPTER XXIV.

THE STRENGTH OF CAST IRON.

347. The Tensile Strength of Cast Iron varies from 15,000 to 35,000 lbs. per square inch, while ordinary foundry irons run from 18,000 to 22,000 lbs.* This strength depends greatly on the size of the specimen as well as on the composition, and on its freedom from internal stress from a too rapid cooling.

The general characteristics of cast iron when tested in tension are shown in Figs. 384 and 385. It will be seen that there is here no well-

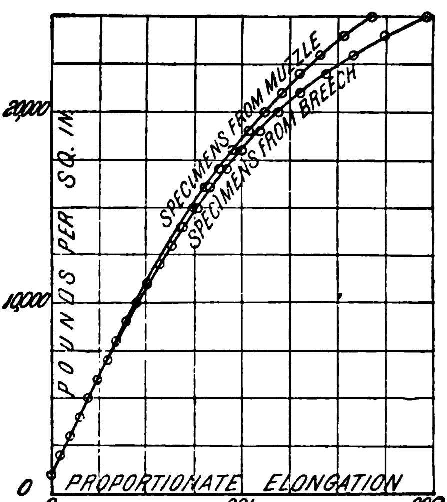

FIG. 384.—Two Stress-diagrams of Cast Iron in Tension, each the average of eleven tests. Average tensile strength = 33,500 lbs. per square inch. (*Wat. Ars. Rep.* 1894.)

defined "elastic limit," and if there be such a point it is very low in comparison with the ultimate strength of the iron. The "apparent elastic limit" falls at about 15,000 lbs. per square inch (Fig. 385), or at about 60%

* When annealed in the malleable process its strength is raised to from 30,000 to 50,000 lbs. per square inch, as shown in Chapter VII, p. 115.

of the ultimate strength, as is found to be the case with wrought iron and rolled steel. The permanent set at this point, while it looks large in Fig.

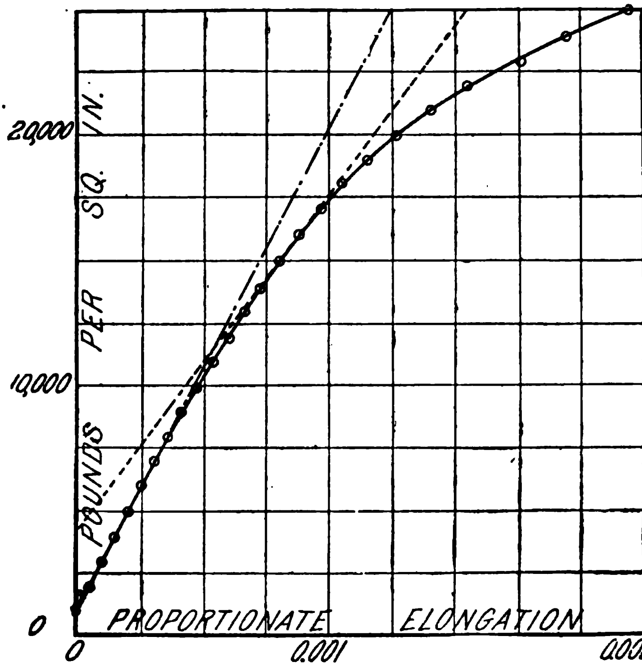

FIG. 385.—Typical Stress-diagram of Cast Iron in Tension, with Location of the "Apparent Elastic Limit," corresponding to a permanent set of 0.0001 of the length. (*Wat. Ars. Tests*, 1892.)

385, is in reality only about $\frac{1}{100}$ of one per cent, or entirely inappreciable. It is safe to specify 25,000 lbs. tensile strength, if great strength is required. Mr. W. J. Keep has shown (Fig. 57, p. 96) that the cross-breaking strength is very largely a function of the size of the test-specimen. As tension-test specimens are usually cast about 1 in. to 1¼ in. in diameter, this variation with size is not so important for tension-test purposes as it might appear from this diagram. The test-specimens are usually turned down, both at the gripped ends and on the reduced portion. Fig. 386 shows a form of cast-iron test-specimen which is intended to dispense with turning altogether; but even here it would be better to turn the gripped ends, to avoid all bending stresses in the grips.

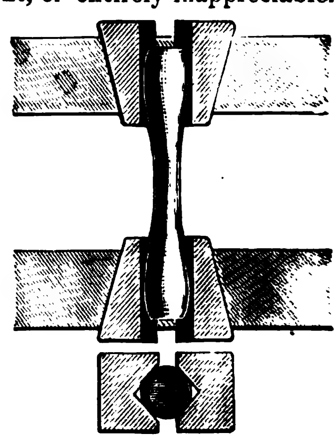

FIG. 386.—Form of Cast-iron Specimen which does not require turning down.

TABLE XXIII.—COMPOSITION AND STRENGTH OF HIGH-GRADE CAST IRONS MADE AT THE FOUNDRY AT THE U. S. ARSENAL AT WATERTOWN, MASS. TEST-SPECIMENS GROOVED. (*Rep.* 1894, p. 247.)

Composition of Charge.	Kind of Furnace.	Carbon. Graphitic.	Carbon. Combined.	Manganese.	Silicon.	Sulphur.	Phosphorus.	Tensile Strength per square inch.	Hardness.
Muirkirk pig .... 35.3 Old 8-inch shell .... 29.4 Heads .... 29.4 Scrap 5.9 100	cupola	2.440	0.900	0.335	1.137	0.118	0.572	27,700	16.07
Muirkirk pig .... 35.3 Shell .... 29.4 Heads .... 29.4 Scrap 5.9 100	do	2.391	0.960	0.342	1.081	0.134	0.505	27,990	15.20
Richmond pig No. 1 ...... 10 Richmond pig No. 2 ...... 10 Salisbury pig No. 4 ...... 15 Salisbury pig No. 4, high 15 Scrap 50 100	do	2.487	0.744	0.461	1.511	0.118	0.521	31,980	17.35
Salisbury pig No. 4 .... 27.5 Salisbury pig No. 4, high 27.5 Scrap 45.0 100	do	3.558	0.606	0.451	1.212	0.125	0.655	32,400	
Salisbury pig No. 4, high 50 Salisbury pig No. 4 .... 50 100	do	2.279	0.366	0.353	1.024	0.118	0.496	34,450	
Salisbury pig No. 4 .... 33.3 Salisbury pig No. 4, high 11.1 Soft pig .... 22.2 Remelted pig 33.3 100	air-furnace	2.492	0.739	0.448	1.231	0.125	0.816	32,980	
Salisbury pig No. 4 .... 25 Salisbury pig No. 4, high 25 Scrap 50 100	cupola	2.393	0.432	0.450	1.090	0.140	0.497	31,110	
Richmond pig No. 1 ...... 11.1 Richmond pig No. 2 ...... 11.1 Salisbury pig No. 4 ...... 16.7 Salisbury pig No. 4, high 16.7 Scrap 44.4 100	do	2.727	0.299	0.462	1.363	0.125	0.477	31,810	15.83
Salisbury pig No. 4 .... 33.3 Salisbury pig No. 4, high 11.1 Soft pig .... 22.2 Remelted pig 33.3 100	air-furnace	2.058	0.778	0.464	1.560	0.115	0.619	29,100	20.47
Salisbury pig No. 4 ...... 20 Salisbury pig No. 4, high 20 Soft pig .... 20 Scrap 40 100	cupola	2.253	0.731	0.458	1.297	0.114	0.491	30,750	18.09

COMPOSITION AND STRENGTH OF HIGH-GRADE CAST IRONS—*continued.*

Composition of Charge.	Kind of Furnace.	Carbon.		Manganese.	Silicon.	Sulphur.	Phosphorus.	Tensile Strength per square inch.	Hardness.
		Graphitic.	Combined.						
Richmond pig No. 1...... 9.4 Richmond pig No. 2...... 9.4 Salisbury pig No. 4....... 9.4 Salisbury pig No. 4, high 9.4 Scrap....... .... 62.5 100	cupola	2.890	0.458	0.388	1.645	0.105	0.487	27,320	
Muirkirk pig........... .. 38.5 Soft pig .... 23.0 Remelted pig............ 38.5 100	air-furnace	2.538	0.979	0.348	1.816	0.130	0.642	26,480	15.67
Salisbury pig No. 4 ..... 9.6 Salisbury pig No. 4, high 9.6 Richmond pig No. 1...... 9.6 Richmond pig No. 2...... 9.6 Soft pig .... 23.1 Remelted pig............. 38.5 100	do	2.770	0.256	0.470	2.444	0.110	0.587	28,010	
Salisbury pig No. 4....... 8.3 Salisbury pig No. 4, high 8.3 Richmond pig No. 1...... 8.3 Richmond pig No. 2 ... . 8.3 Soft pig... .. .. 20.0 Remelted pig 46.7 100	do	2.751	0.357	0.435	1.908	0.095	0.420	29,120	11.08
Salisbury pig No. 4... . 33.3 Salisbury pig No. 4, high 11.1 Soft pig ... ... 22.2 Remelted pig... .... 33.3 100	do	2.538	0.634	0.355	1.222	0.090	0.766	28,520	21.04
Salisbury pig No. 4 .. ... 33.3 Salisbury pig No. 4, high 11.1 Soft pig.. .... 22.2 Remelted pig............ 33.3 100	do	2.577	0.185	0.361	1.146	0.115	0.762	31,020	
Salisbury pig No. 4....... 33.3 Salisbury pig No. 4, high 11.1 Soft pig... .... 22.2 Remelted pig............ 33.3 100	do	2.116	0.640	0.450	1.419	0.125	0.678	31,140	17.44
Salisbury pig No. 4....... 22.5 Salisbury pig No. 4, high 22.5 Scrap.................. 55.0 100	cupola	2.825	0.479	0.361	1.062	0.076	0.288	32,010	16.82
Salisbury pig No. 4...... 8.3 Salisbury pig No. 4, high 8.3 Richmond pig No. 1...... 8.3 Richmond pig No. 2... .. 8.3 Soft pig. .... 20.0 Remelted pig..... .... 46.7 100	air-furnace	2.481	0.687	0.454	1.175	0.120	0.678	31,990	

The tensile strength and the relative hardness of various high-grade compositions are given in Table XXIII.

348. The Compressive Strength of Cast Iron varies from 60,000 to 200,000 pounds per square inch as shown in Fig. 55, p. 94. In Fig. 387 is shown a

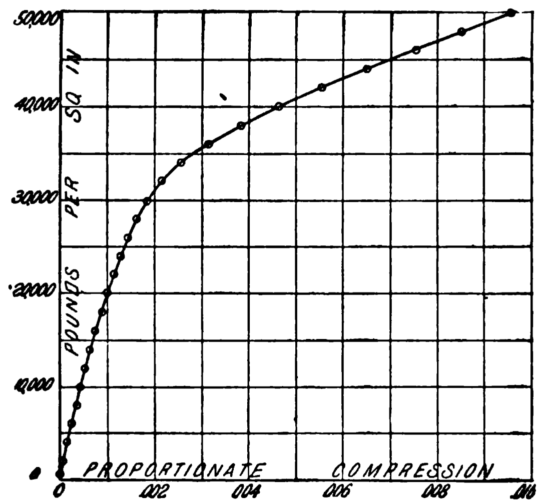

FIG. 387.—Average Results of Twenty-two Tests of Cast Iron in Compression, from B. L. 12-in. Rifle-mortars. (*Wat. Ars. Rep.* 1894, p. 105.)

stress-diagram in compression, plotted from the average results from twenty-two tests of gun-iron. As these were made on specimens 10.5 inches long, having a sectional area of 1 sq. in., they all failed by triple flexure, or as columns, at an average value of 63,000 pounds per square inch, the actual crushing strength not having been found. (The tensile strength averaged 33,500 pounds per square inch.)

In Fig. 388 are shown Bauschinger's results on four kinds of cast iron, as follows:

No. 1 is composed wholly of coke pig iron.
" 2 " " " " charcoal pig iron.
" 3 " 90% coke-pig and 10% steel.
" 4 " 80% coke-pig and 20% steel.

The tests were made to determine the time-effect in testing cast iron, there being two specimens of each kind in both tension and compression, with one of which a rest of one minute was allowed after each increment of load, and with the other a rest of five minutes. No time-effect was discovered, however, and here the mean curves only are shown.

In all these cases it will be observed that no well-marked elastic limit can be identified, although the curves are plotted to a very large scale of deformations.

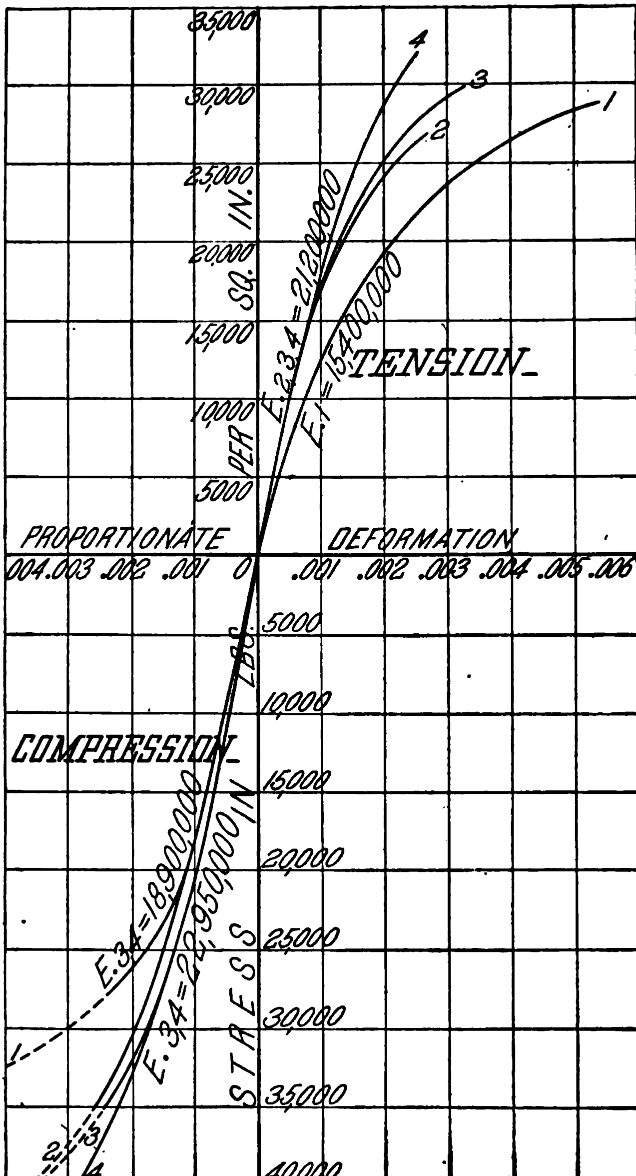

FIG. 388.—Stress-diagrams on Four Kinds of Cast Iron. Each Curve the mean of two tests. (Bauschinger's Communications, vol. xx, Plate XII.)

349. The Cross-breaking Strength of Cast Iron is in general from one and one half to two and a quarter times its strength in tension on solid rectangular sections. The cause of this was discussed in Chapter V. In Fig. 389 are shown autographic stress-diagrams of four kinds of cast iron made on Keep's standard test-bars ½ in. square and 12 in. long. They all showed the same strength of 450 lbs. at the centre, giving a computed modulus of rupture of 64,800 pounds per square inch. It will be noted that their deformations under like loads are very different, thus giving rise to enormous differences in their strength to resist shock. Thus their resistances to shock, as determined by the total areas of their stress-diagrams, are respectively 10.0, 21.5, 28.9, and 35.1 inch-pounds per cubic inch.* These four tests were selected to show the necessity of observing the deflections as well as the loads, if resistance to shock is to be found. These diagrams also show that no great error is made in the case of cast-iron if the area of the stress-diagram in cross-bending be assumed to be equal to one half the product of the breaking load into the total deflection. This is the common rule for cast iron. This half-product, divided by the volume or weight of the specimen between the supports, gives the shock-resisting modulus in inch-pounds per cubic inch or per pound of metal as the case may be, and is independent of the particular dimensions *except that the smaller the cross-section the greater the strength-modulus*, as is conclusively shown in Keep's curves in Fig. 57, p. 96. Higher shock-resisting moduli will be obtained, therefore, on small (thin) sections than on large ones, and the only safe rule is to learn by trial what products to expect and to demand for given sizes of test-specimens and given grades of iron. (See Fig. 57, p. 96.)

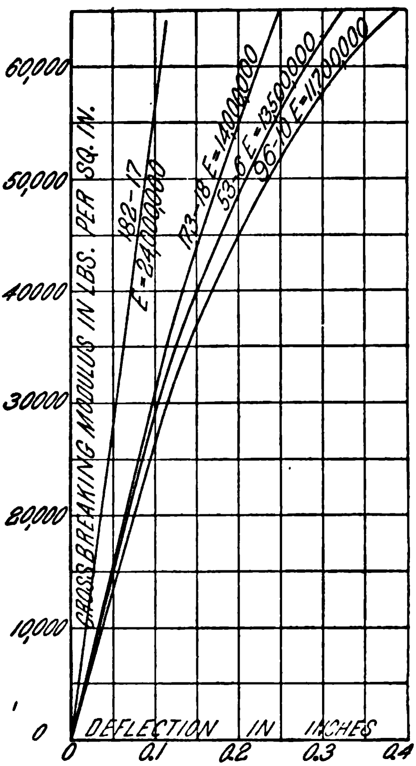

FIG. 389.—Cross-breaking Autographic Stress-diagrams of Four Kinds of Cast Iron all having the Same Static Strength. (Keep, *Tr. Am. Soc. Mech. Engrs.*, vol. XVII, 1896.)

The committee of the American Society of Civil Engineers recommended (1896) a cross-breaking test of cast iron in which $f = 36{,}000$ lbs. on a bar

* These can be taken out per pound of metal if preferred.

2 in. by 1 in. tested flatwise on a 24-in. base, with a deflection of 0.3 in. This corresponds to only 6¼ inch-pounds per cubic inch.

Mr. W. J. Keep has shown * that the 1-in. square bar gives more uniform results than any other size, because at this size the effects of varying percentages of silicon are not appreciable, as is shown by Fig. 57, p. 96. In reducing the great number of tests on cast iron (some 500 in all) in sizes from ½ in. square to 4 in. square made for the Committee on Methods of Testing Materials of the American Society of Mechanical Engineers, Prof. Benjamin has found that the strength of all sizes of cast-iron bars can be related to that of the 1-in.-square bar 12 in. long by the following formula: †

$$W = k\frac{b^{0.83}h^{1.89}}{l^{1.058}}, \qquad (1)$$

where W = breaking load on the centre of the bar;
k = breaking load for a bar 1 in. square and 12 in. long;
b = breadth of the bar in inches;
h = height of the bar in inches;
l = length of the bar *in feet* (multiples of 12 in., which is the length of the standard bar.)

The theoretical relation is $W = k\frac{bh^2}{l}$.

350. The Modulus of Elasticity of Cast Iron varies quite as much as its strength. In this respect it is anomalous, since for all rolled iron and steel the modulus of elasticity varies only about five per cent from its mean value for strength variations, in the case of steel, of several hundred per cent. There is probably a pretty definite relation between the strength of cast iron and its modulus of elasticity, the stronger iron having the higher modulus of elasticity, that is, it is stiffer. At least this relation is very clearly shown in the curves in Fig. 55, p. 94. In general this modulus varies from 10,000,000 to 30,000,000, but for ordinary foundry iron it may be taken at from 12,000,000 to 15,000,000, or about one half that of wrought iron and rolled steel.

It may be well here to call attention again to a ready method of reading the modulus of elasticity from any of the tension- or compression-stress diagrams given in this book. Where the loads are given in pounds per square inch, and the deformations in percentages, or proportionate parts of the length, then by observing where the tangent to the curve at the origin (or the curve itself if it is straight so far) crosses the ordinate which marks a deformation of 0.001, one has only to read the corresponding stress per square inch and multiply it by 1000. The modulus of elasticity of cast iron is approximately the same in tension, in compression, and in cross-bending.

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* *Trans. Am. Soc. Mech. Engrs.*, vol. XVII. (1896) p. 681.

† *Ibid*, p. 692.

351. Kirkaldy's Results.—In Table XXIV are given the results of 469 tests of cast iron in each of the three ways, tension, compression, and cross-bending, all on identical material in each case. The averages of all are:

Tensile strength	25,000 lbs. per square inch
Compressive strength	121,000 " " " "
Cross-bending modulus.	38,000 " " " "
Quality-coefficient	6.5 in.-lbs. per cubic inch.

The mere fact that these specimens were submitted to Mr. Kirkaldy for testing implies that the mixtures were better than are commonly used in foundry practice, and yet these average results could readily be reached in any good foundry.

The small ratio of the cross-breaking modulus of rupture to the tensile strength, this having an average value of but 1.52, is due to the great depth (2 in.) of the transverse test-bar. The small value of the "quality-coefficient" is doubtless due to the same cause. Otherwise all these irons would be classed as comparatively brittle.

As this "quality-coefficient" was not taken out by Kirkaldy, and as it is probable that only breaking strength was specified, it is quite probable that high strength has been attained in this material at the expense of resilience or shock-resistance. The ratio of the cross-breaking modulus to that in tension is also lower with less flexibility. The modulus of elasticity was not computed for any of these tests, and as only the final deflections are given in the published report, it cannot now be computed from the tabular matter.

352. Shrinkage Stresses.—The shrinkage of cast iron after it crystallizes is so great that, if not provided for, it causes excessive deformations which may develop very great stresses, even to rupture. The heavier or the thicker the casting the greater are these shrinkage stresses. These have been studied in the case of cast-iron guns, and one such analysis is shown in Fig. 390. Here the metal was over 11 inches thick. The outer and inner surfaces cooled first, and the subsequent shrinkage of the interior put these parts in compression. But since the total internal stress across any diametral section must be zero, there being no external force acting, it follows that the total tensile stress must equal the total compressive stress. These were all found directly by cutting off a zone included between two transverse sections, and by cutting this up into a series of concentric rings as shown by the dashed lines in Fig. 390. Before cutting these, four diameters of each ring were carefully measured, and these same diameters were again measured after cutting out. An increase in mean diameter indicated an initial compression, and *vice versa*, the initial stresses being found from the equation

$$f = \lambda E,$$

where f = stress in pounds per square inch;
λ = proportionate change in circumference;
E = modulus of elasticity of the material.

TABLE XXIV.—SUMMARY OF RESULTS OF TESTS ON CAST IRON IN TENSION, COMPRESSION, AND CROSS-BENDING, ON IDENTICAL MATERIAL.

From Kirkaldy's Report, 1891 (Report T T).

Total Number of Tests from One Foundry.	Grade.	Tension Strength in Pounds per Square Inch.	Compression.		Cross-bending.		
			Strength in Pounds per Square Inch.	Ultimate Deformation, Per Cent.	Computed Stress on Outer Fibre in Pounds per Square Inch.	Ultimate Deflection in Inches.	* Resistance to Shock or "Quality-factor" in Inch-pounds per Cubic Inch.
151	Highest	32,821	141,632	6.65	47,710	.32	8.84
	Mean	**26,165**	**122,279**	**9 26**	**38,020**	**.27**	**5.94**
	Lowest	16,250	103,165	12.20	25,820	.21	3.14
74	Highest	27,614	124,251	....	37,390	.32	6.23
	Mean	**24,303**	**117,242**	....	**33,840**	**.26**	**5 09**
	Lowest	19,311	109,682	....	27,260	.21	3.31
58	Highest	28,740	131,912	13.80	43,540	.40	1.00
	Mean	**24,148**	**115,572**	**9.98**	**39,550**	**.36**	**8.24**
	Lowest	17,698	93,759	4.45	33,840	.33	6.46
46	Highest	30,630	137,165	11.80	46,460	.33	8.87
	Mean	**23,339**	**105.918**	**11.95**	**36,190**	**.36**	**7.54**
	Lowest	12,688	66,363	12.70	34,240	.38	5.33
15	Highest	29,782	138,496	10.40	46,650	.36	9.72
	Mean	**22,727**	**116,833**	**10.66**	**44,460**	**.35**	**8 99**
	Lowest	15,580	83,307	7.90	42,860	.32	7.94
15	Highest	26,040	132,857	....	36,100	.28	5.85
	Mean	**23,925**	**123,044**	....	**34,760**	**.26**	**5.22**
	Lowest	22,711	113,233	....	31,360	.24	4.35
15	Highest	25,708	123,531	9.20	40,660	.36	8.47
	Mean	**23,129**	**116,356**	**9 12**	**39,700**	**.36**	**8.27**
	Lowest	17,617	105,258	7.35	38,400	.34	7.56
13	Highest	27,644	122,708	8.55	39,290	.39	8.19
	Mean	**24,321**	**116,538**	**8.64**	**32,780**	**.29**	**5.50**
	Lowest	19,188	104,281	7.00	26,930	.21	3.27

* This "quality-coefficient" is a measure of the resistance to shock, or it is the area of the stress-diagram in inch-pounds per cubic inch, found by multiplying the

SUMMARY OF RESULTS OF TESTS ON CAST IRON—*continued.*

Total Number of Tests from One Foundry.	Grade.	Tension Strength in Pounds per Square Inch.	Compression.		Cross-bending.		
			Strength in Pounds per Square Inch.	Ultimate Deformation, Per Cent.	Computed Stress on Outer Fibre in Pounds per Square	Ultimate Deflection in Inches.	*Resistance to Shock or "Quality-factor" in Inch-pounds per Cubic Inch.
13	Highest	26,502	175,950	4.05	54,140	.45	14.10
	Mean	**21,711**	**123,336**	**5.49**	**40,750**	**.34**	**8.02**
	Lowest	16,090	103,859	6.45	27,070	23	3.60
10	Highest	25,176	127,988	7.55	36,050	.32	6.68
	Mean	**24 298**	**125,962**	**7.03**	**34,750**	**.29**	**5.83**
	Lowest	23,511	120,874	6.40	31,870	.25	4.61
10	Highest	30,316	136,266	12.25	40,460	.30	7.02
	Mean	**29,268**	**133,682**	**12.61**	**38,020**	**.27**	**5.94**
	Lowest	28,436	129,524	13.90	34,080	.21	4.14
10	Highest	30,018	130,145	12 10	39,550	.34	7.78
	Mean	**29,472**	**127,714**	**11 90**	**38,400**	**.32**	**7.11**
	Lowest	27,501	122,155	11.75	37,050	.29	6.22
10	Highest	28,416	140,542	6.60	38,780	.31	6.96
	Mean	**27,763**	**138,054**	**6.54**	**37,250**	**.28**	**6.03**
	Lowest	26,851	135,577	6.60	35,330	.27	5.52
10	Highest	25,520	127,708	7.80	34,960	.32	6.46
	Mean	**24,803**	**121,593**	**7.71**	**33,020**	**.28**	**5.35**
	Lowest	23,435	110,405	6.60	30,190	.26	4.54
10	Highest	28.518	129,603	9.10	40 370	.35	8.18
	Mean	**27.914**	**126,701**	**9.70**	**38,060**	**.32**	**7.05**
	Lowest	27,276	124,415	9.15	35,470	.26	5.34
9	Highest	33,616	143,939	....	46,990	.41	11.15
	Mean	**29,202**	**137,804**	....	**43,490**	**.39**	**9.81**
	Lowest	19,046	124,410	....	43,490	.31	6.82

breaking load by the final deflection and dividing by twice the volume of the bar. It was not computed by Kirkaldy. All these bars were 2 in. × 1 in. × 36 in. long, tested edgewise, the tension and compression specimens being cast on the same pattern. The great depth of these bars makes this quality-factor and also the modulus of rupture run low as compared to tests on thinner specimens.

In this way the stress-diagrams shown in Fig. 390 were computed and drawn by the author from the data furnished in the original report. It indicates that the interior surface was under an initial compressive stress of some 7000 lbs. per square inch, the outer surface of some 13,500 lbs. per square inch; while the interior was under a tensile stress of some 2000 lbs. per square inch. Evidently the tension and compression areas on these diagrams must equal each other. This is a very simple illustration of such shrinkage stresses, because of its simple and symmetrical form. In complex forms it would be impossible to study or predict the character of these

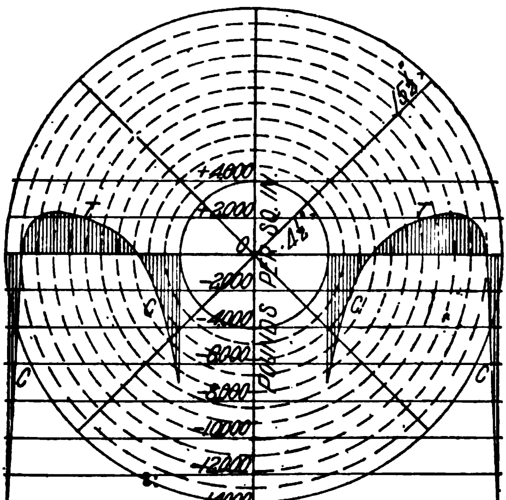

FIG. 390.—Showing the Shrinkage Stresses in Cast-iron Cannon 11 in. thick. (*Wat. Ars. Rep.*)

stresses. They are evidently less when all parts are made of approximately the same thickness.

353. Strength of Cast Iron Increased by Shocks.—Mr. A. E. Outerbridge has shown* that castings which have been subjected to a great number of shocks or blows are from 10 to 15 per cent stronger under a static load and over 20 per cent stronger under impact than they are before receiving such treatment. He attributes this result to a sort of molecular rearrangement by which the cooling stresses are relieved. In other words, such treatment is equivalent to an annealing process.—This is probably a general fact and true for all kinds of castings, although this remains to be proved.

354. Strength of Malleable Cast Iron.—For the experimental results of tests of the strength of malleable cast iron see Art. 82, p. 114.

355. Cast-iron Pipes and Columns.—In estimating the actual strength of cast-iron pipes subjected to internal pressure, a great source of uncertainty

* *Trans. Am. Inst. Min. Engrs.*, Pittsburg Meeting, 1896.

lies in the probable unequal thickness of the metal in the upper and lower sides when cast. There is a great tendency of the core to rise from the buoyancy of the liquid iron, and since no core, however strong, can be absolutely rigid, it must be assumed that it does always lift somewhat at the centre, however strongly it is held at the ends. As a matter of fact, the cores are not usually very rigid, so that such pipes are apt to be very unequal in thickness on the opposite sides, as shown in Fig. 391. The regular bell-and-spigot water- and gas-pipes are now all cast vertically, and hence this

Fig. 391.—Actual Section of an 8-in. Cast-iron Steam-pipe. (From *The Locomotive*, Oct., 1896.)

danger is largely obviated, but flange-pipes, such as are used for steam purposes, are cast horizontally. Any great inequality in thickness can be found by rolling the pipe down inclined ways and noting the irregularity of motion

Cast-iron columns, such as are used in buildings, are also cast horizontally, and are subject to this same contingency. These may be bored to determine thickness, but pipes cannot be examined in this way. It is such undiscoverable faults as this which form the greatest objection to the use of cast iron for these purposes. For other defects in cast-iron columns see Plate III.

CHAPTER XXV.

THE STRENGTH OF WROUGHT IRON.

356. The Tensile Strength of wrought iron along the grain varies from 45,000 to 55,000 pounds per square inch. It is greater in small rods and thin plates than in large bars and thick plates, the material remaining the same. This is shown in Fig. 392, where the same material has been rolled into bars from ⅝ in. to 2 in. in diameter, the tensile strength varying from 52,000 in the smaller to 47,500 pounds per square inch in the larger sizes.

The Elastic Limit is more dependent on the thinness of the final section than on the tensile strength, as is well brought out in Fig. 392. Here the apparent elastic limit varies from 40,000 pounds per square inch in the ⅝-in. rods to 23,000 pounds per square inch in the 2-in. rods, and is almost identical with the "yield-point." This increase in the elastic limit with increased reduction in the rolls always occurs with both wrought iron and steel, but it is much more pronounced with wrought iron. The true elastic limit of wrought iron is nearly always much lower than the apparent elastic limit. In Fig. 392 it is found from 5000 to 7000 lbs. lower in every case. In mild steel these two limits are almost identical.

The Percentage of Elongation in 8 in. varies from 5% to 25% when tested in the direction of the fibres, depending on the quality of the material, the reduction of area averaging about 50% more than the elongation. The elongations recorded in Fig. 392 were all taken on a length of 20 in., which somewhat reduces the percentage, especially for the smaller sections.

357. The Tensile Strength across the Grain is always much less than along the grain in the case of wrought iron, while with steel there is no appreciable difference. Very few tests of wrought iron across the fibres are to be found on record, but the author has often observed in his own practice that it is very much less in this direction than parallel to the direction of rolling. In Prof. Bauschinger's Communications, vol. II, we find an elaborate study of this subject on wrought-iron boiler-plates from eight different sources, some of them having been taken from boilers which had exploded. From this report we have—

1. From eight tensile tests along and eight across the grain, from an exploded boiler, the ratio of lateral to longitudinal strength was 0.74.

2. From a wrought-iron plate from another exploded boiler eight test-

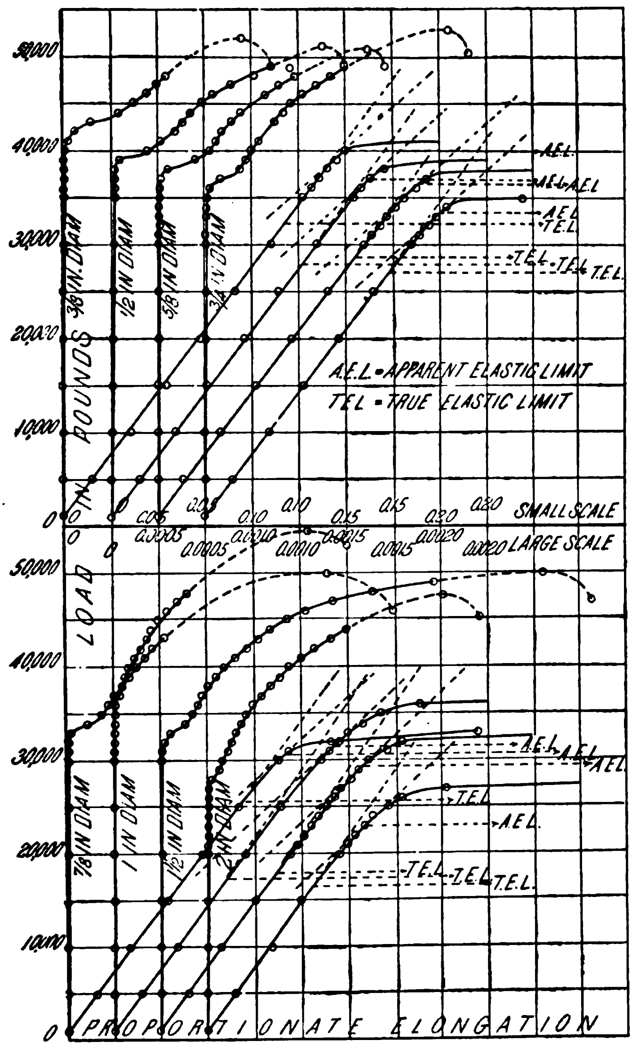

Fig. 392.—Stress-diagrams (in tension) of Wrought-iron Bars of varying Diameters, all rolled from the same Material. All Elongations measured on a Length of 20 in. The " Apparent Elastic Limit " falls from 2% to 10% lower than the indicated "Elastic Limit " in the original Report; it varies from 28,000 in the 2-in. to 40,000 lbs. in the $\frac{3}{8}$-in. specimens; and it marks a point where the permanent set is less than 0.0001 of the length of the specimen. Each diagram is the average of from 3 to 6 tests. (*Wat. Ars. Rep.* 1838.)

specimens were cut in each direction, giving a mean ratio of lateral to longitudinal strength of 0.71.

3. On six other new plates from as many different sources he obtained ratios of 0.76, 0.62, 0.92, 0.90, 0.76, and 0.83.

The average value of all these is **0.78.**

In short, we may fairly affirm that the ultimate tensile strength of wrought iron transverse to the direction of the rolling is only about three fourths of its strength parallel to this direction.*

The author is credibly informed that the best English Yorkshire (Lowmoor) iron plates are always rolled from "puddled lumps" 12 in. square, which correspond to muck-bars, these being piled so as to cross their grain, and in this way the final plates are nearly as strong transversely as they are longitudinally. It is claimed that for an ultimate strength of 51,500 pounds per square inch, with an elongation of 16% longitudinally, it shows an ultimate strength of 45,000 pounds per square inch and an elongation of 12% transversely. As this material costs about four times as much as the best mild-steel plates, its use for all purposes where forging and welding are not required is rapidly declining.

358. Tensile Strength of Wrought Iron as Affected by Pulling Speed and by Length of Reduced Section.—In Table XXV are given the results of a series of very careful tests to determine the effects of speed and of the length of the reduced section on the tensile strength of wrought iron. It will be observed that the ultimate strength is somewhat increased by very rapid testing, while the elongation and reduction are not appreciably affected by this average range of speed from 15 sec. to 8.5 min. The strength is also much greater for very short reduced sections than for longer ones. Similar results on steel bars are shown in Fig. 426.

359. The Compressive Strength of Wrought Iron, like that of any of the ductile metals, must be regarded as the "apparent elastic limit" or "yield-point." Here the material buckles out of shape, and if the specimen has appreciable length, failure at once follows. If this be allowed, then it may be seen at once, from Fig. 392, that the same puddle-ball, rolled to different sections, will show compressive strengths anywhere from 26,000 to 40,000 lbs. per square inch. Since wrought-iron columns are built up of structural forms which have been rolled to thin sections, from ¼ to ⅝ in. in thickness, it follows that such material will have a "yield-point" or "apparent elastic limit" of from 30,000 to 40,000 lbs. per square inch. Since, also, the amount of reduction in the rolls has a less effect on the elastic limit of mild steel, it follows that the yield-point, and hence the compressive strength, of a wrought-iron column may be about equal to that of a steel member built up of similar sections, although the ultimate strength of the steel in tension may be 25 per cent higher than that of the wrought iron. This variation of the compressive strength (yield-point) of wrought-iron columns with the

* The author has been unable to find the data for plotting a stress-diagram of wrought iron across the grain.

TABLE XXV.—TESTS OF WROUGHT IRON TO SHOW EFFECT OF SLOW AND OF RAPID FRACTURES ON SPECIMENS OF VARYING LENGTH.

Sixteen specimens taken from the same bar of iron 1⅝ in. square, all reduced to a diameter of 1.008 in. or to a sectional area of 0.80 sq. in. Specimens marked from *A* to *P* consecutively, as cut from the bar. (From *U. S. Wat. Ars. Rep.* 1887, p. 924.)

Marks.	Length.	Elastic Limit per Square Inch.	Ultimate Strength per Square Inch.	Duration of Test.	Gauged Length.	Elongation in Gauged Length.	Contraction.	Final Load per Square Inch on Ruptured Section.
	Inches.	Pounds.	Pounds.		Inches.	Per cent.	Per cent.	Pounds.
A	Grooved	....	57,200	6 min.			32.4	72,090
B	Grooved	....	59,250	6 sec.			32.4	....
C	0.80	27,375	49,380	6 min.			49.1	71,260
D	0.80	....	50,750	8 sec.			37.1	....
E	1.60	28,500	47,730	10 min.	1	•56.0	47.6	78,280
F	1.60	....	49,130	13 sec.	1	50.0	46.2	....
G	2.40	30,125	47,980	8 min.	2	39.0	43.2	61,680
H	2.40	....	49,000	14 sec.	2	41.5	49.1	....
I	3.20	29,625	47,070	10 min	3	39.0	49.1	77,150
J	3.20	....	48,120	15 sec.	3	36.0	47.6	....
K	4.80	29,875	46,860	10 min	4	32.0	43.2	70,260
L	4.80	....	48,000	18 sec.	4	32 8	47.6	....
M	6.40	29,750	47,450	9 min.	6	25.0	49.1	80,590
N	6.40	....	48,250	20 sec.	6	32.0	47.6	....
O	8.00	28,500	45,650	9 min.	8	25.9	41.7	66,950
P	8.00	....	46,000	30 sec.	8	25.9	41.7	....
Mean of slow tests =			48,670	8.5 min.		36.1	44.4	73,500
Mean of quick tests =			49,810	15.5 sec.		36.4	43.7	....

thickness of the sections of which the member is composed accounts for a large portion of the discrepancies found in the results of tests on wrought-iron columns in commercial sizes (see Fig. 297, p. 364). Thus Tetmajer found for short columns composed of four angle-irons, 2.4 in. × 2.4 in. × ¼ in. in thickness, the average strength of the wrought-iron columns was 38,000 lbs. per square inch of net section, while that of similar mild-steel columns was but 36,000 lbs. per square inch. The ultimate tensile strength of the wrought iron was 50,000 lbs. per square inch, while that of the steel was 61,000 lbs. per square inch. With angles 0.4 in. thick the steel columns had an ultimate strength of 37,500 lbs. per square inch, while the wrought-iron columns showed only 32,300 lbs.* With sections ⅜ in. thick and less, therefore, there is probably little difference between the strength of wrought-iron and soft-steel columns.

360. The Shearing Strength of Wrought Iron.—The most elaborate investigation ever made on wrought iron, so far as the author is aware, was that made by Bauschinger and reported in vol. II of his *Communications.* He made several hundred tests of the shearing strength of wrought-iron plates from seven different sources, finding the shearing resistance in two

* *Communications*, vol. IV. pp. 141, 149, and 155.

directions on each of the three of the principal planes, as shown in Fig. 393. As there was a general agreement in the relative strength on these planes, only the averages of a portion of the tests are given in the diagram. In general, we may say that the shearing strength across the thickness of the plate, either with or across the grain, is about 80 per cent of the tensile strength, while if the external forces lie in the plane of the plate, and be applied on the

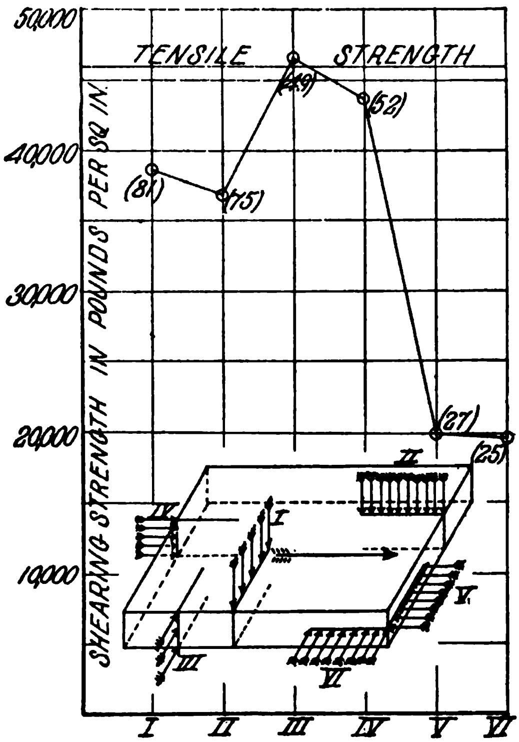

FIG. 393.—Shearing Strength of Wrought Iron on the Six Principal Planes, as compared to its Tensile Strength. The numbers indicate the number of results averaged. The direction of rolling is indicated by the large arrow. (Bauschinger's *Communications*, vol. II.)

planes of shear perpendicular to the plane of the plate, the shearing strength is about the same as the tensile strength. The shearing resistance on a plane parallel to the plane of the plate is less than 45 per cent of the tensile strength.

361. The Effects of Stressing Wrought Iron beyond its Elastic Limit is to raise this limit, and also to greatly increase the ultimate strength after a period of rest. Thus in Fig. 394 are shown the results of tests on three wrought-iron bars 3 in. × 1 in. Here the apparent elastic limit on the second test (computed on the original cross-section) is much greater than the original ultimate strength, and almost equal to the ultimate strength on

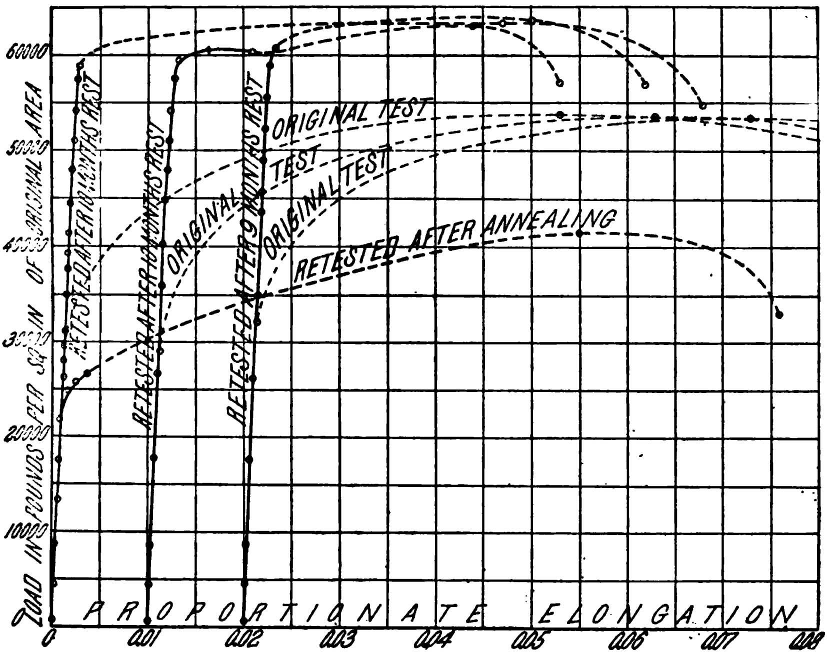

FIG. 394.—Wrought-iron Bars, 3 in. by 1 in. retested. First test gave El. lim. = 30,000; ult. str. = 53,700; % elong. = 16 on 100 in. Ends of broken bars retested and loads computed per sq. in. of original section. Second Elongation taken on 50 in. and per cent elongation computed on the new gauged length. (*Rep. Wat. Ars.* 1882.)

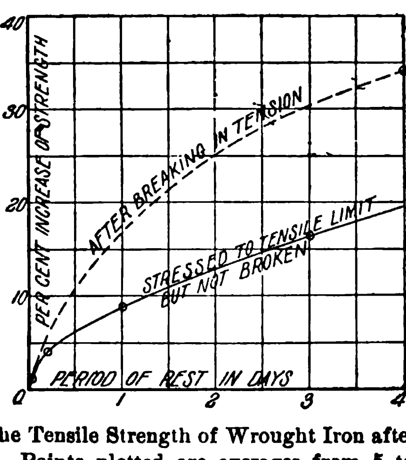

FIG. 395.—Increase in the Tensile Strength of Wrought Iron after having been Stressed to the Tensile Limit. Points plotted are averages from 5 to 15 tests each. (*Rep. U. S. Test Board*, 1881, vol. I. pp. 111 and 115.)

the second test. After annealing, however, the material showed a much lower elastic limit and ultimate strength than it had before it was stressed at all. Stretching the material somewhat beyond the elastic limit, but not to failure, would show less marked results. If little or no time is allowed to elapse between the tests, there is no permanent increase in the strength, but the rate of increase in strength with time after stretching to its tensile limit is well shown in Fig. 395.

When any ductile metal is stressed beyond its normal elastic limit in either tension or compression it loses its perfect elasticity under the opposite kind of stress. Thus for wrought iron, Fig. 396, has been constructed by

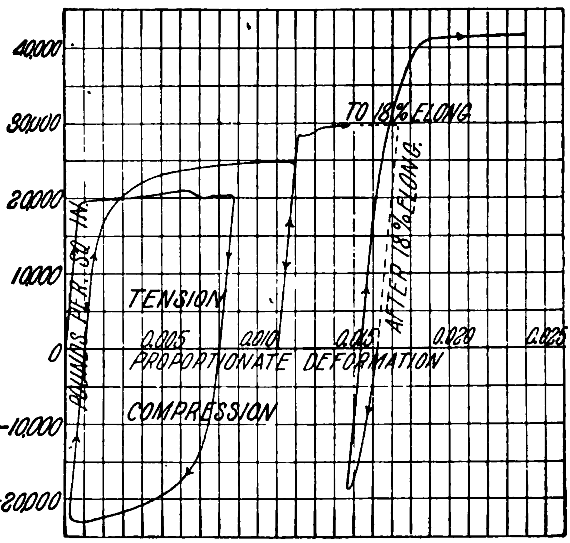

FIG. 396.—Showing that a small Permanent Set of Wrought Iron from either Tension or Compression greatly reduces the Elastic Limit under the Opposite Stress. (Gray, in *The Digest of Physical Tests*, vol. I. p. 232 (1896).)

plotting consecutively a series of autographic diagrams taken by Prof. Gray.* Although at first the permanent set given to the specimen in tension was less than one per cent of its length, yet when this was relieved and a compressive stress applied it was shown that the specimen was no longer perfectly elastic in compression, but gave a continuously curving stress-diagram. It was then compressed back to its original length, and a tensile stress applied, when it was found to be no longer perfectly elastic in tension. It was now permanently elongated about one per cent of its length, and the load removed and again imposed, and it was found that the specimen was now perfectly elastic in tension up to the limit of its previous loading and somewhat higher, but when it was now elongated 18 per cent it was no

* Reported in *The Digest of Physical Tests*, vol. I. p. 206.

longer perfectly elastic in either tension or compression. Similar results are shown on steel in Figs. 436 and 437.

362. The Strength of Wrought-iron Chains.—The hundreds of tests on wrought-iron chains given in the Report of the U. S. Test Board, 1881, vol. I, show that the ultimate strength of chains may be taken at 1.6 that of the iron from which the links are made. It also appears from these tests that open links are somewhat stronger than studded links, though the open-link chains take a permanent set earlier than the studded links. It is thought, however, that open-link cables would foul more readily than the studded cables. The elastic properties of open-link chains made of 1-in. and $\frac{7}{8}$-in. iron are shown in Fig. 397, where the tests have been carried to the proof-load only, this being such as to give to the chains a permanent set of about two per cent of their length. They now become perfectly elastic to 20,000 and 15,000 lbs. respectively, and are also about five times more stiff, or rigid, than they were at first. All chains are improved by this treatment, while it also discovers any very poor welds the chain may have.

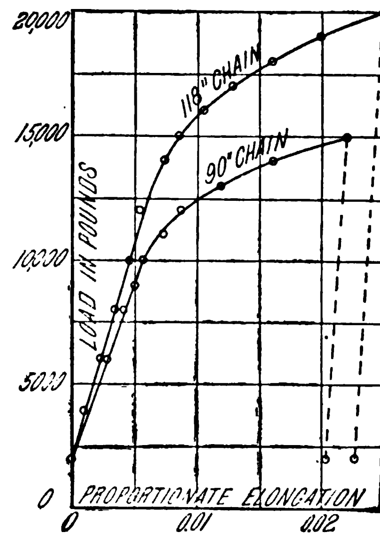

FIG. 397.—Proof Tests of Chains with Open Links. The 118-in. chain made of 1-in. iron, and the 90-in. chain of $\frac{7}{8}$-in. iron. (*Wat. Ars. Rep.* 1894.)

CHAPTER XXVI.

THE STRENGTH OF STEEL.

356. The Tensile and the Compressive Strength of Steel of various percentages of carbon are well shown in Figs. 399 to 404.* A study of these

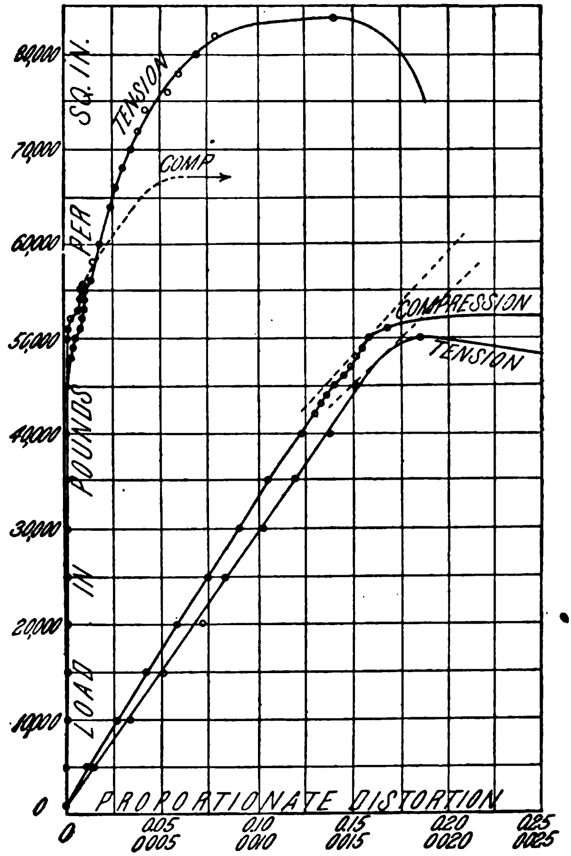

FIG. 398.—Compression and Tension Tests on Midvale Steel Bars. Compression tests on bars 1 in. diam. and 5 in. long; tension tests on bars 0.56 in. diam. and 5 in. long. (*Wat. Ars. Rep.* 1889.)

* In all these figures the loads are given in pounds per square inch, though this is not always so stated.

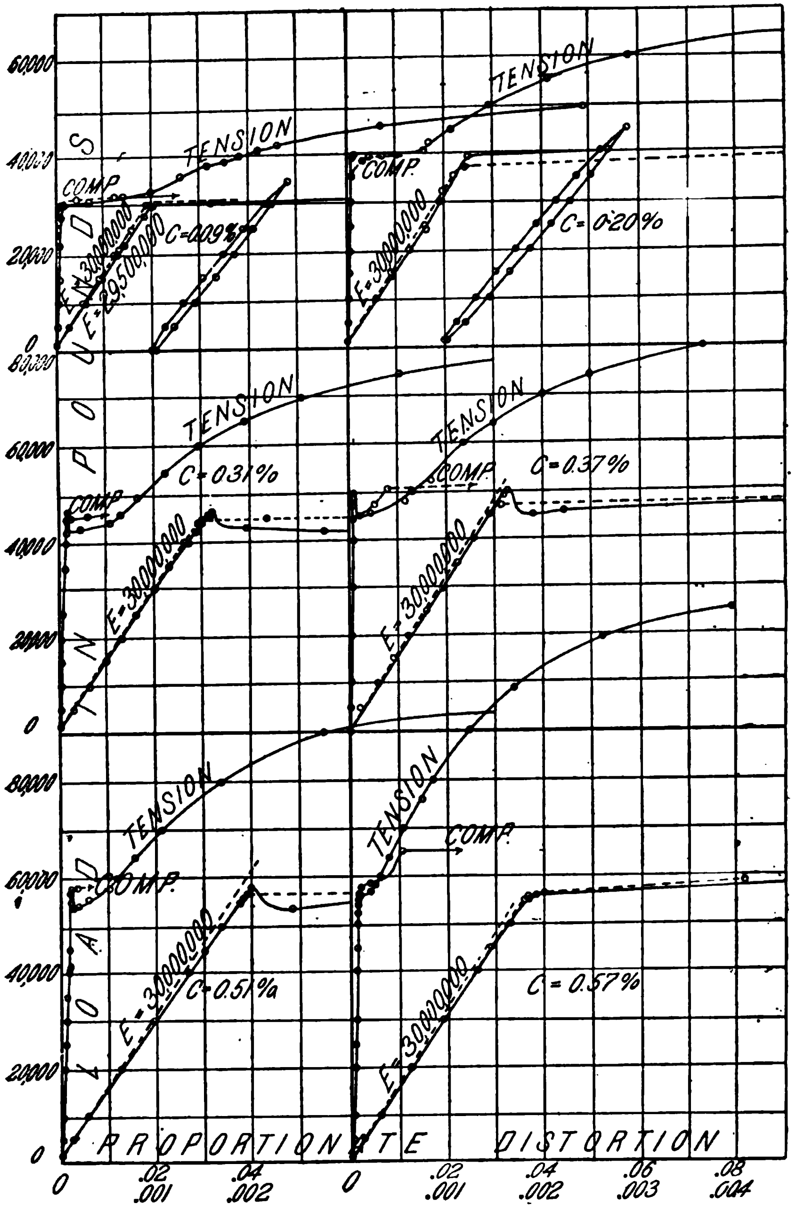

FIG. 399.—Compression and Tension Stress-diagrams of Steel Bars of Varying Percentages of Carbon. Compression specimens 12 in. long and 1 in. in diam. $\left(\frac{l}{r} = 33\right)$, with flat ends. Tension specimens same diam. with a gauged Length of 30 in. All specimens turned from open-hearth steel bars $1\frac{1}{4}$ in. in diam. (*Wat. Ars. Rep.* 1886 and 1887.)

figures, all of which are typical and characteristic, will lead to the following conclusions:

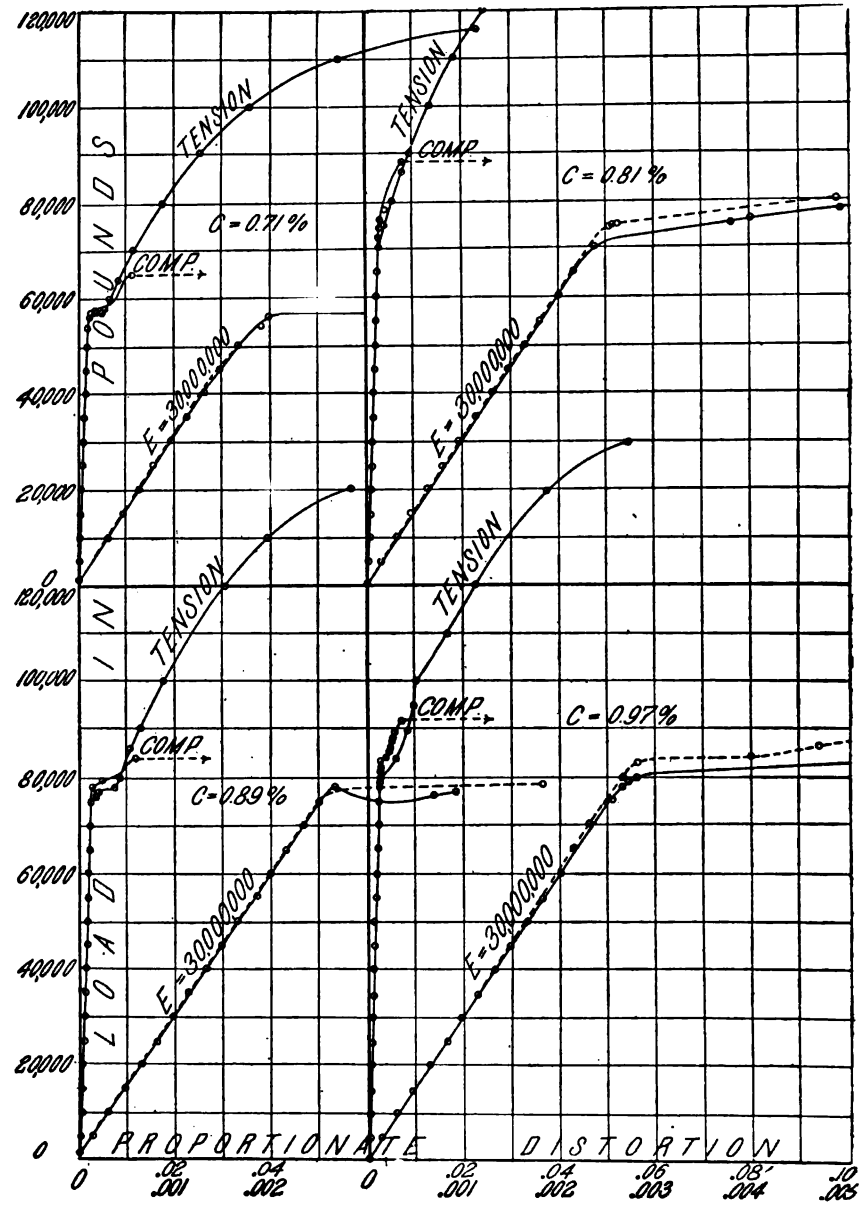

FIG. 400.—Steel Tests supplementary to those of Fig. 399.

1. The tensile strength varies from 55,000 lbs. for 0.1 per cent carbon to 150,000 lbs. per square inch for 1.0 per cent carbon.

2. The "apparent elastic limit" is found between 60 and 70 per cent of the ultimate strength.

3. The "apparent elastic limit" in compression is practically the same as that in tension.

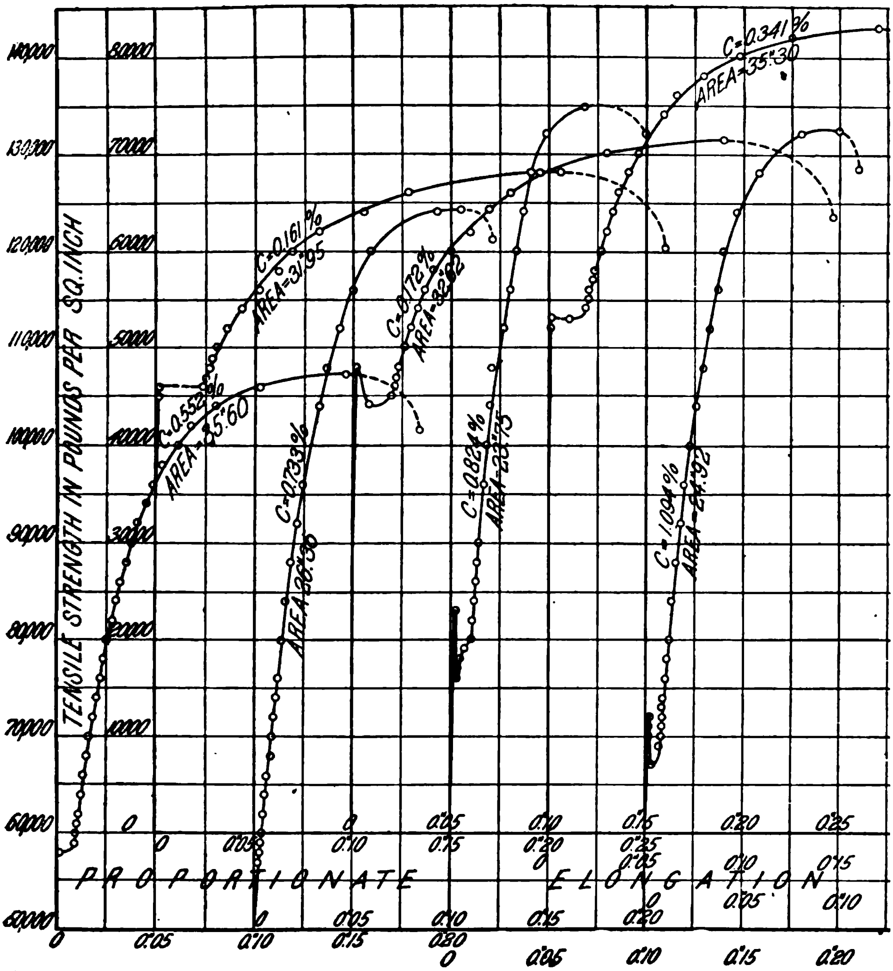

FIG. 401.—Tension Stress-diagrams of Gautier Steel Bars which were used in the Endurance Tests on Rotating Shafting subjected to Reversals of Stress. Percentages of carbon and areas of the stress-diagrams in square inches are given on the curves. (*Rep. Wat. Ars.* 1894.)

4. The modulus of elasticity in compression is slightly greater than that in tension, and in both cases it is practically independent of the percentage of carbon and of the ultimate strength.

5. The ultimate strength in compression is practically equal to the "apparent elastic limit" (Fig. 294, p. 360).

6. In the mild and medium steels (carbon 0.2 to 0.6 per cent) there is a very decided drop in the stress-diagram after reaching the "apparent elastic limit" often as much as 5000 or 6000 lbs. per square inch.

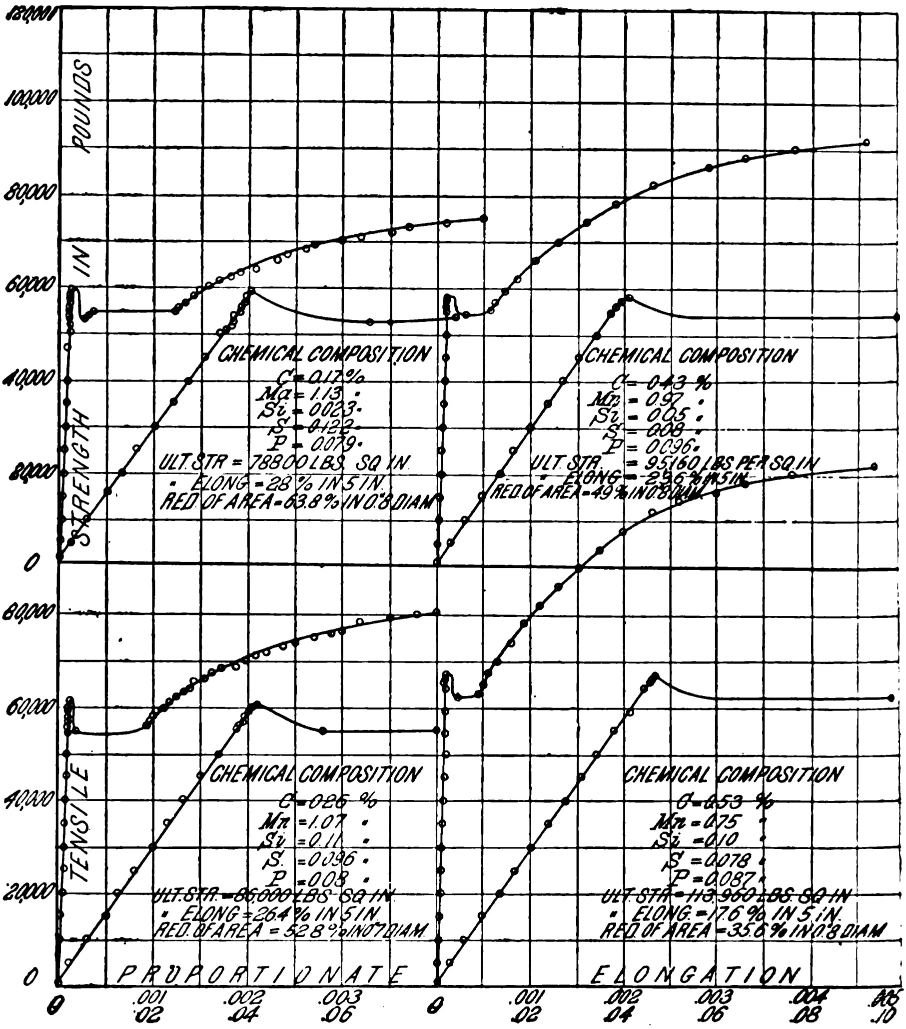

FIG. 402.—Tension Stress-diagrams (incomplete) of Steel Bars used for Endurance Tests of Rotating Shafting. Average product of ultimate strength by percentage of elongation is 2,180,000. (*Wat. Ars. Reps.* 1889 and 1891.)

7. The coefficient of expansion decreases with the increase in carbon, its average value being about 0.0000065 per degree F. (Fig. 294).

8. The high carbon-steels are greatly softened, the tensile strength lowered, and the ductility increased by annealing (Fig. 404).

357. The Effect of Thickness on the Mechanical Properties of Structural Steel.—In Figs. 406, 407, and 408 are shown the effect of thickness (bars and angles) on the mechanical properties of structural steel. Thus from

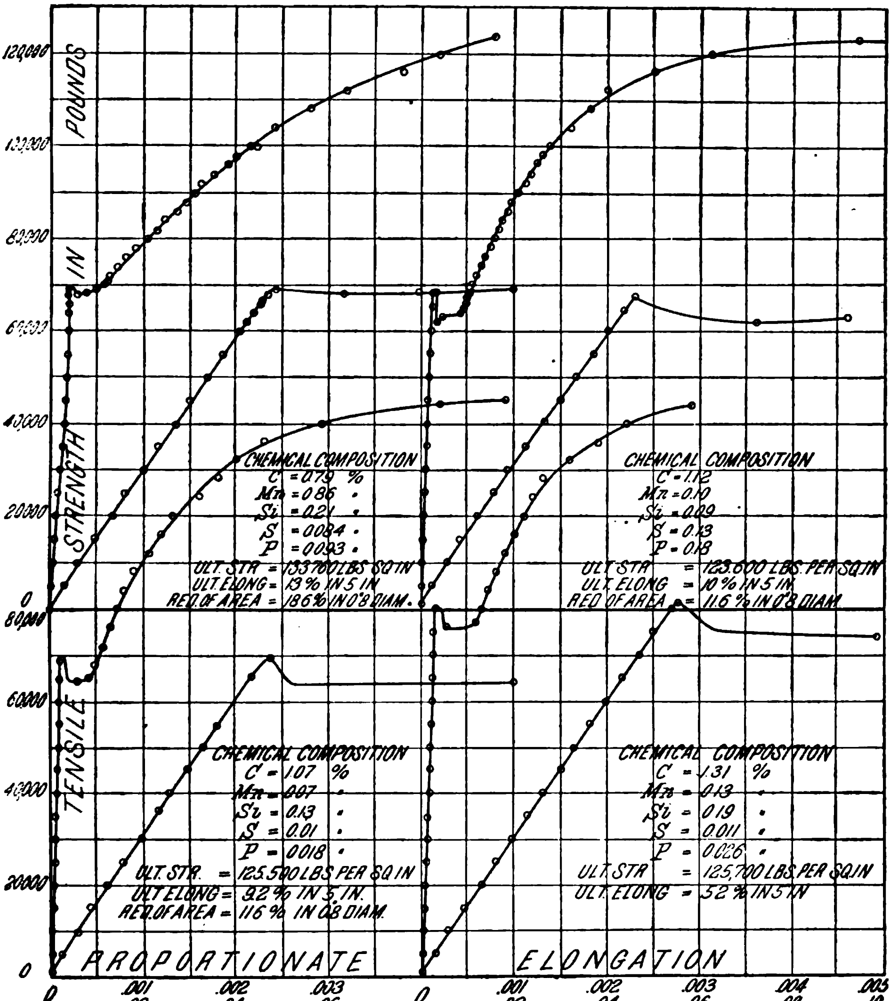

FIG. 403.—Tension Tests of Steel Bars used for Endurance Tests of Rotating Shafting. (*Wat. Ars. Rep.* 1889 and 1891. Supplementary to Fig. 402.)

Fig. 406, where the thickness ranges by eighths of an inch from $\frac{3}{8}$ to $\frac{7}{8}$ inch, we may see—

1. The ultimate strength is nearly constant.
2. The apparent elastic limit varies from 41,000 at $\frac{3}{8}$ to 37,800 lbs. per square inch at the $\frac{7}{8}$-in. thickness.

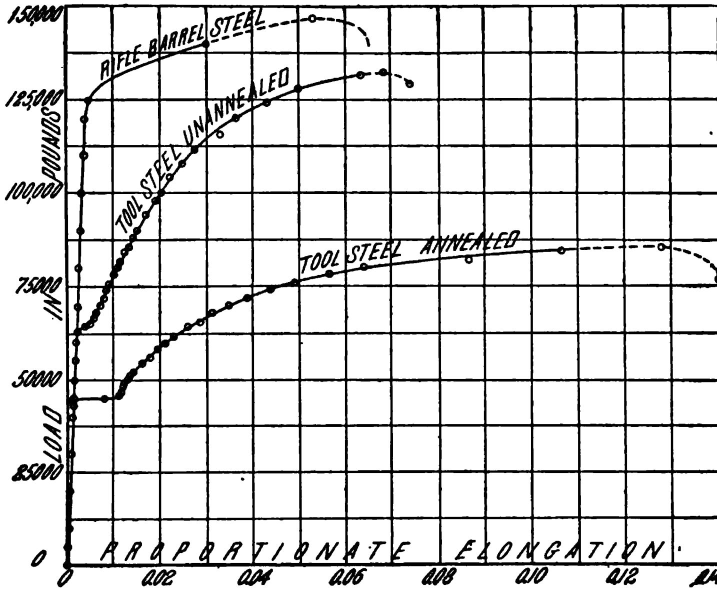

FIG. 404.—Tension Stress-diagrams of Three Grades of High-carbon steel. (*Wat. Ars. Rep.* 1894.)

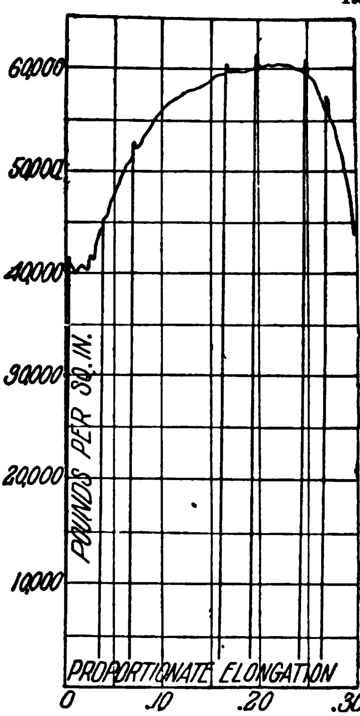

FIG. 405.—Autographic Stress-diagram of Rivet-steel in Tension, showing Effect of Removing the Load. (M. Dupuy, in *An. d. Ponts et Chaussées*, Pl. I. 1895.)

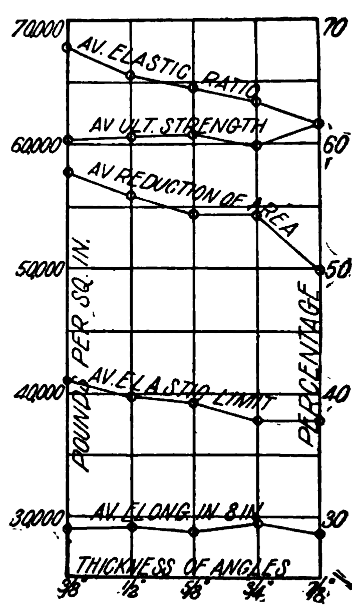

FIG. 406.—Effect of Thickness on the Mechanical Properties of Acid Open-hearth Steel Angles. (Campbell's *Structural Steel*, p. 202.)

3. The percentage of elongation in 8 in. is nearly constant.

4. The reduction of area varies from 58 per cent at ⅛ in. to 50 per cent at ⅝ in. thickness.

5. The elastic ratio: $\frac{\text{Elastic limit}}{\text{Ultimate strength}}$ varies from 68 per cent at ⅛ in. to 61.5 per cent at ⅝ in. thickness.

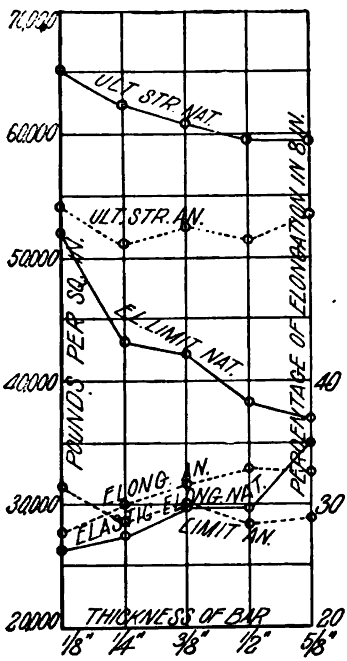

FIG. 407.—Effect of Thickness on the Mechanical Properties of Mild Steel, Natural and Annealed. (Campbell's *Structural Steel.*)

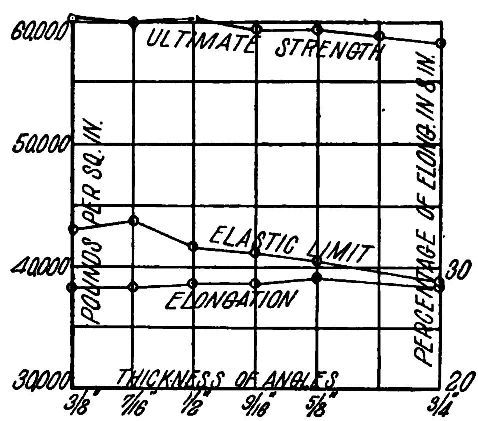

FIG. 408.—Effects of Thickness on Bessemer Steel Angles. Each point is the mean of fifty results. (Campbell's *Structural Steel*, p. 199.)

From Fig. 409 it may be seen that the variation in ultimate strength and in the elastic limit for different thicknesses is much greater when the metal leaves the rolls at a dull-red heat. Here the thickness ranges from ⅛ in. to ⅝ in., and the elastic limit for normal rolling varies from about 50,000 lbs. per square inch at the ⅛-in. thickness to 39,000 lbs. at a ⅝-in. thickness. When leaving the rolls at a dull-red heat, however, the elastic limit for the ⅛-in. thickness was 57,500 lbs., while for the ⅝-in. thickness it was only 42,000 lbs. per square inch.

In general the apparent elastic limit rises as the thickness of section diminishes. Since wrought-iron and steel columns are built up from comparatively thin sections of metal (generally from ¼ to ½ in. in thickness), and as the ultimate strength of these is dependent wholly on the apparent elastic limit, and not at all on the ultimate strength, it is necessary to evaluate this elastic limit for the particular thicknesses of sections used, rather than from

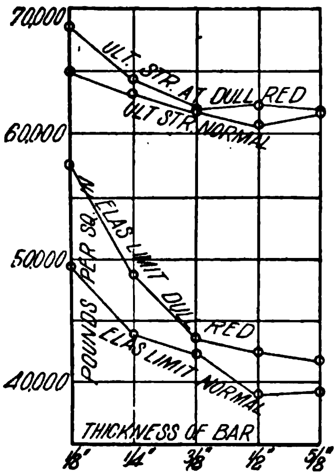

FIG. 409.

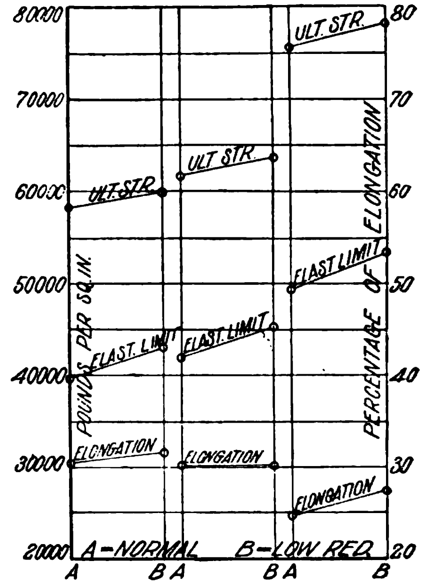

FIG. 410.

FIG. 409.—Influence of Thickness on Mechanical Properties when the Percentage of Reduction in Rolling is constant, and when the Last Passage in the Rolls was at the Normal and at a Dull Red Heat respectively. (Campbell's *Structural Steel.*)

FIG. 410.—Showing the Effect of finishing Three Grades of Open-hearth Steel Bars at a Low Red Heat. (Campbell's *Structural Steel*, Table 70.)

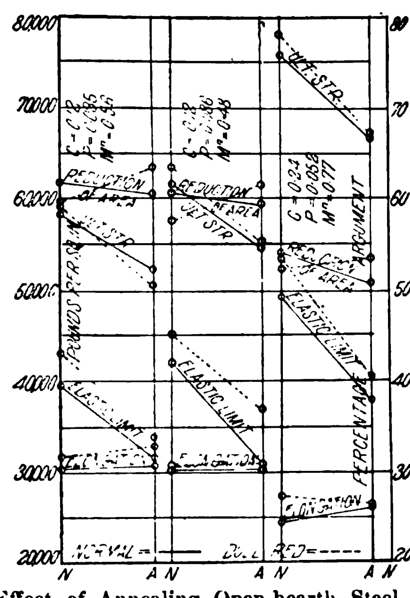

FIG. 411.—Showing Effect of Annealing Open-hearth Steel Bars 2 in. × ⅜ in. when Rolled Originally at a Normal and at a Low Red Heat. (Campbell's *Structural Steel*, p. 213.)

special test-bars, which are usually not less than ⅞ in. in thickness. (See Table XXV, p. 503. for the comparison between the results obtained on the preliminary ⅞-in. billet test-specimen and those from the specimens cut from rolled bars and plates of various thicknesses from the same ingot.)

The comparatively small variation in the elastic limit (and other properties) shown in Fig. 408 is due to the fact that all were rolled from the same sized ingot, and thus the thinner sections had more work done upon them. When the proportionate reductions are the same in each case the differences are very much greater, as shown in Fig. 409. These differences almost wholly disappear on annealing, as shown by Fig. 407.

358. Effects of Finishing at a Low Red Heat.—As shown by Figs. 409 and 410, the effect of finishing at a low red heat is to somewhat increase the ultimate strength and the elongation, and to greatly increase the elastic limit. This last increase is as much as from 8 to 10 per cent. From Fig. 411 it appears that while annealing lowers both ultimate strength and elastic

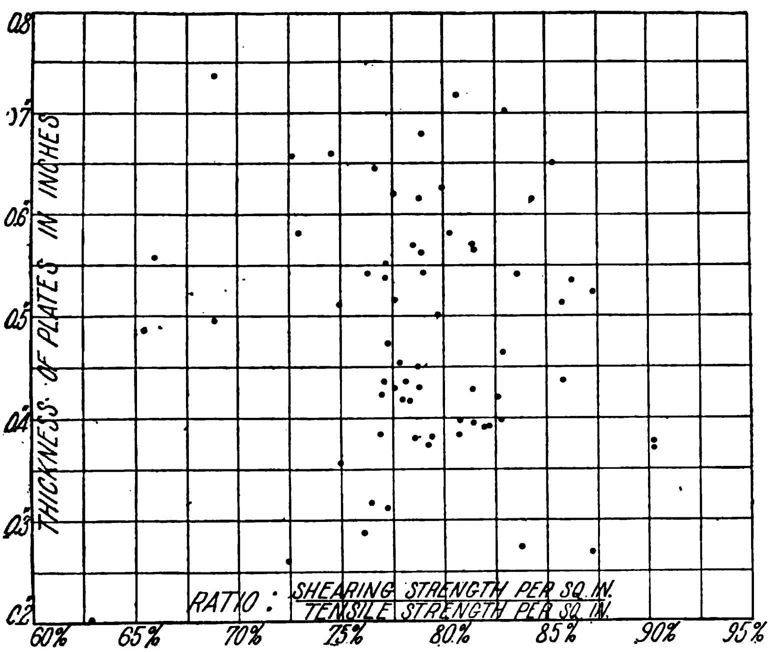

FIG. 411*a*.—Showing the Absence of any Law of Relationship between Shearing Strength and Tensile Strength of Steel Plates when the Shearing Strength is determined by Punching Tests. (Experiments made at Washington University by Messrs Condron, Harrington, and Norton. See a full account of these in *Engr. News*, vol. XXXII (1894), p. 164.)

limit, it does not appreciably increase the elongation, neither does it bring the "normal" and "dull red" specimens much nearer together in the matter of the elastic limit. It might fairly have been assumed that annealing

would have removed the effects of rolling at a low heat, as it always does the effects of cold working, but it has not done so in this instance.

358a. Punching Tests of Steel Plates.—It has been claimed * that structural steel may be tested by punching in place of the ordinary tension tests. This subject was very fully investigated at the Washington University Testing Laboratory, as the subject for a thesis, by Messrs. Harrington and Norton, in 1894, assisted by Mr. T. L. Condron, C.E., and it was concluded that no definite relationship could be established between the results of punching and of tension tests on steel plates of varying quality and thickness. The tests were made with autographic stress-diagrams of every test, and hundreds of tests made on steel plates of known chemical composition and tensile properties. Fig. 411*a* is here given as one of many such studies made of the results, all going to show that the punching tests could not be employed as tests of acceptance under any set of specifications.†

359. Effects of Quenching and of Annealing on Structural Steel.—It is not generally known that quenching from a bright cherry-red heat has a

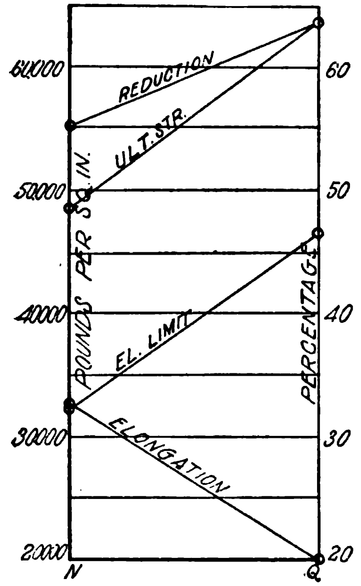

FIG. 412.

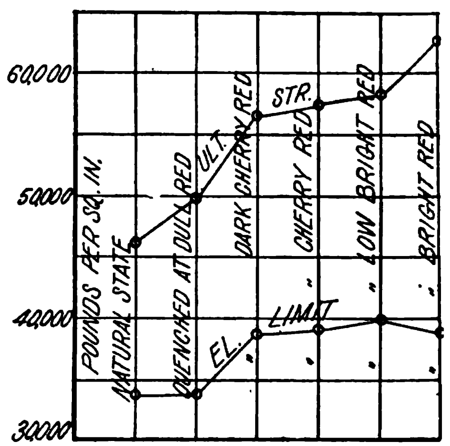

FIG. 413.

FIG. 412.—Average Effects of Quenching Soft Steel (0.10 C.) from a Dull Red Heat. Each point the mean of six tests. (Campbell's *Structural Steel*, p. 52.)

FIG. 413.—Effects of Quenching a Very Low Carbon (0.057 C.) Steel at Different Temperatures. (Campbell's *Structural Steel*, p. 53.)

marked effect on the softest grades of open-hearth steel. That it has is shown by Figs. 412 and 413, where a steel having a normal tensile strength

* By Alfred E. Hunt before the World's Engineering Congress, Chicago, 1893. *Trans. Am. Soc. Civ. Engrs.*, vol. xxx. p. 181.

† See an illustrated article by Mr. Condron in *Engr. News*, vol. xxxii. p. 164.

of 48,500 lbs. per square inch (Fig. 412) is raised to 63,500 lbs. per square inch by quenching. The apparent elastic limit is raised from 32,000 to 46,000 lbs. per square inch; the elongation is lowered from 32 to 20 per cent in 8 in.; and the reduction of area is raised 55 to 63 per cent.* This reveals also the difference between the elongation and the reduction of area as characteristics of steel. The highest grade of steel wire, having a strength of 200,000 lbs. per square inch and upward, and having an elongation of but one or two per cent, will commonly show a reduction of area of over 60 per cent. That is, the stretch largely occurs at the necked-down portion only. The cold-bending test is a test of the reduction of area rather than of the elongation. In Fig. 413 the effects are shown of quenching soft open-hearth steel from various temperatures from a dull red to a bright red heat. These two figures both show that the softest steel is greatly changed by quenching, but since the reduction of area has been raised its capacity for making short bends has been increased.

When the cold-bending test is specified, after quenching, therefore, it would appear from Fig. 412 that this soft material is better able to stand

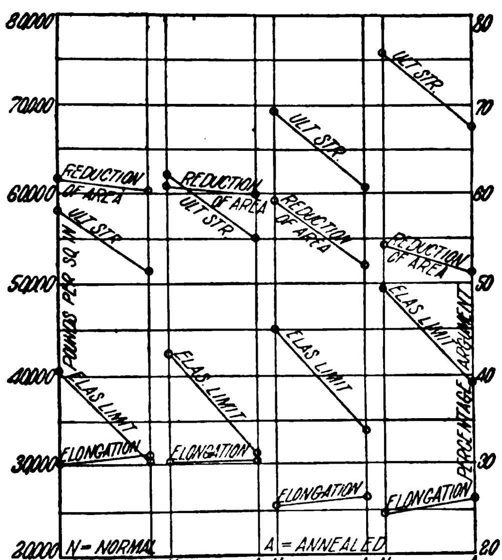

FIG. 414.—Effect of Annealing 2 in. × ⅜ in. Open-hearth Steel Bars of Different Degrees of Hardness. (Campbell's *Structural Steel*, p. 210.)

this test than it was in its normal condition. This matter should be proved by direct bending tests, however, before being accepted as true. If this proves to be true, then quenching greatly improves the steel for all purposes.

The annealing process seems always to reduce the ultimate strength and the elastic limit for all grades of steel, the lowering of the latter, as shown

* The chemical composition was C = 0.105; Mn = 0.343; P = 0.009; S = 0.024.

in Fig. 414, being about 25 per cent. The elongation is very slightly increased and the reduction of area somewhat decreased. By the annealing process, therefore, about one fourth of the effective strength of a steel member is sacrificed, while its cold-bending capacity is probably reduced. It would seem, therefore, that it should be practised only when necessary to remove severe internal stresses produced by cold working.

360. The Billet Test is Characteristic of the Final Rolled Bars and Plates.—Mr. Gus C. Henning has shown * that specimen billets rolled or forged from the sample of the steel dipped from an open-hearth furnace to test its quality give results which are truly characteristic of the products rolled from that heat. The tests of these sample billets are commonly called "heat tests," since one such is made for each heat. If the results of these tests agree closely with the tests on specimens cut from the structural forms, bars, and plates which are rolled from this material, then these latter may be omitted and reliance placed wholly upon the heat tests. The results of 221 such corresponding sets of tests are given in Table XXV.

This table also contains results of tests of specimens cut from annealed bars and plates. In all the specimen tests on rolled and annealed bars and plates an extensometer was used, reading to 0.0001 in. by micrometer-screw with electric contact, as shown in Fig. 271. The true elastic limits were therefore carefully and accurately determined. This was not done with the heat or billet tests, so that no comparison can be made on this score. The yield-points were observed by the "drop of the beam," which is a quite accurate method with this quality of material if the test is not made too rapidly.

361. The Distribution of the Elongation over an 8-inch specimen $1\frac{1}{8}$ inches in diameter is shown in Fig. 415. This shows that the stretch is nearly uniform until the maximum load is reached, after which it begins to neck down. From this on to final rupture the stretch is almost wholly confined to the necked-down portion. Thus while the elongation near the ends was only 21 per cent and the average elongation was about 28 per cent, the elongation at the plane of rupture was 75 per cent.

The Reduction of Area of rectangular specimens is difficult to obtain accurately, on account of the resulting curved outlines arising from the greater contraction in the middle portions. This action is well shown in Fig. 416. If the reduced section is calipered at about one fourth the width from each side, the area resulting from taking the product of these two dimensions will be insignificant.

362. The Compressive Strength is the Elastic Limit.—Table XXVI contains the results of some of the most careful experiments ever made on the compressive strength of steel bars (see also Fig. 294). These were made by Mr. Chas. A. Marshall, C.E.† These results show a practical identity in the apparent elastic limit, or yield-point, in tension and compression, and also

* In *Trans. Am. Soc. Mech. Engrs.*, vol. XIII.

† See foot-note p. 341.

TABLE XXV.—COMPARISON OF RESULTS OF TESTS OF SPECIMEN BILLETS AND OF SPECIMENS CUT FROM FINAL ROLLED FORMS.

Size of Bar.	Kind of Test.	Elastic Limit.	Yield-point	Tenacity.	Per cent Elongation in 8″	Per cent Reduction in 8″	Modulus of Elasticity.	No. of Tests Averaged
				TESTS OF BARS.				
7″ × 7/8″	Billet		47,267	73,440	23.6	39.5	30,191,000	3
	Rolled	35,867	39,006	71,540	25.1	51.6	29,799,000	3
	Annealed	37,850	39,083	69,990	25.3	54.2	31,290,000	3
7″ × 1⅛″	Billet		45,666	70,845	23.3	41.7	29,498,000	4
	Rolled	33,622	39,334	71,102	23.2	44.7	30,837,000	5
	Annealed	35,060	40,140	67,930	26.3	57.0	31,567,000	2
7″ × 1 3/16″	Billet		44,498	70,392	23.3	41.3	30,793,000	3
	Rolled	29,650	36,620	69,750	24.1	48.9	29,939,000	2
	Annealed	33,110	40,035	70,860	25.2	53.7	31,410,000	2
7″ × 1¼″	Billet		45,010	71,035	22.6	37.1	29,150,000	2
	Rolled	34,725	38,255	72,108	22.8	40.2	29,889,000	4
	Anaealed	38,125	40,725	69,800	24.6	42.4	31,444,000	2
7″ × 1 5/16″	Billet		46,103	69,820	24.0	41.8	29,900,000	3
	Rolled	31,550	36,635	67,795	26.0	55.0	30,921,000	2
	Annealed	35,688	39,860	68,154	25.8	54.8	31,451,000	4
7″ × 1⅜″	Billet		45,508	70,820	24.2	42.5	30,127,000	14
	Rolled	32,890	37,480	71,561	21.6	38.5	30,166,000	19
	Annealed	35,470	39,579	70,450	23.4	44.7	30,934,000	11
7″ × 1 7/16″	Billet		46,319	71,395	23.1	38.3	29,850,000	10
	Rolled	29,606	34,882	67,560	24.2	46.0	30,430,000	12
	Annealed	34,169	37,834	68,070	24.6	48.0	30,510,000	11
7″ × 1½″	Billet		47,350	71,595	22.0	37.9	29,840,000	1
	Rolled	33,730	38,000	70,090	26.2	48.8	30,528,000	1
	Annealed	34,620	38,500	69,840	27.5	57.5	30,528,000	1
7″ × 1 11/16″	Billet		45,698	70,665	23.7	41.5	29,500,000	7
	Rolled	32,082	36,315	71,269	23.3	43.5	30,759,000	11
	Annealed	37,170	39,152	69,688	25.4	50.1	31,268,000	10
7″ × 1¾″	Billet		44,950	68,770	25.0	40.0	28,285,000	1
	Rolled	38,100	41,040	74,060	19.8	42.0	31,647,000	1
	Annealed	35,000	37,550	65,420	22.9	35.0	30,746,000	1
7″ × 1 13/16″	Billet		45,740	72,330	23.6	38.1	29,455,000	1
	Rolled	29,080	31,510	66,180	26.2	54.9	32,479,000	1
	Annealed	32,100	36,640	73,670	23.7	50.4	31,302,000	1
7″ × 1⅞″	Billet		46,320	71,630	20.8	34.9	28,535,000	1
	Rolled	29,000	32,750	71,370	25.0	52.9	28,420,000	1
	Annealed	33,650	39,060	71,280	24.5	58.7	31,078,000	1
Average	Billet		45,869	71,020	23.8	39.5	29,594,000	Total No. 161
	Rolled	33,327	36,819	70,365	23.9	47.5	30,484,000	
	Annealed	35,167	39,013	69,596	24.9	46.0	31,127,000	
				TESTS OF PLATES.				
1/2″	Billet		47,700	80,966	20.5	35.8	30,605,000	5
	Rolled	47,048	48,120	86,572	19.7	36.3	30,061,000	5
	Annealed	39,523	43,698	75,155	24.7	47.3	29,901,000	9
5/8″	Billet		47,737	82,113	20.6	33.9	29,690,000	6
	Rolled	45,895	48,008	81,753	19.9	40.7	30,763,000	6
	Annealed	39,792	42,066	74,624	22.1	47.2	30,591,000	9
3/4″	Billet		51,936	81,818	21.1	38.8	29,612,000	5
	Rolled	44,692	47,643	79,127	21.8	42.3	30,879,000	5
	Annealed	38,267	40,067	72,961	22.9	47.8	31,167,000	7
13/16″	Billet		50,130	84,970	19.7	30.8	29,220,000	1
	Rolled	45,580	48,280	88,230	19.0	31.2	29,188,000	1
	Annealed	41,730	43,590	77,010	25.8	50.8	30,528,000	1
Average	Billet		49,376	82,467	20.5	34.8	29,782,000	Total No. 60
	Rolled	40,804	48,018	83,920	20.1	37.6	30,223,000	
	Annealed	39,828	42,355	74,938	23.9	48.3	30,722,000	

that the ultimate strength of short bars is the apparent elastic limit. They also show the lessened elastic limit and elongation for the larger sizes, all being rolled from the same billet, the ultimate strength not varying much. Thus for a reduction of ultimate strength from ¾ in. to 2½ in. diameter of

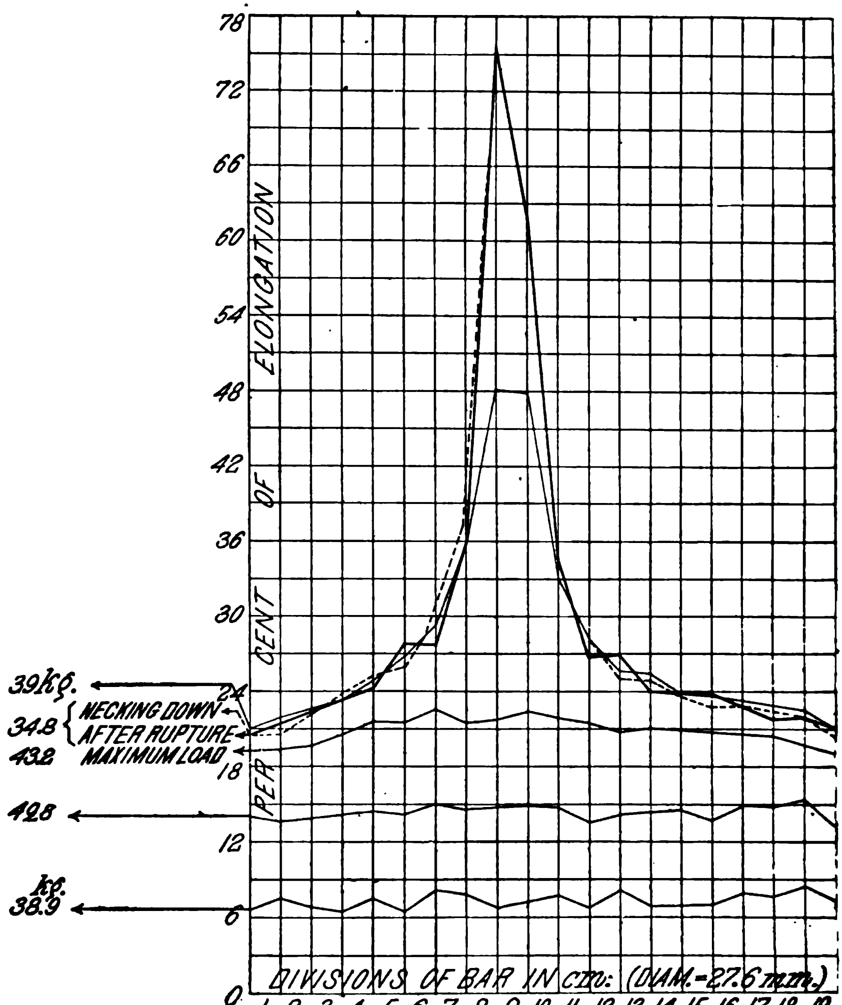

FIG. 415.—Showing the Distribution of the Elongation on (62,000-lb.) Steel Specimen 8 in. long and 1⅛ in. in diameter. (*Fr. Com. Rep.*, vol. III, Pl. III.)

5.4 per cent, we have a reduction in the elastic limit of 20 per cent, and in the percentage of elongation of 60 per cent, the ratio of length to diameter remaining constant.

363. The Elastic Limit in Compression marks the beginning of lateral flowing of the metal, and the stress under which this action begins depends

TABLE XXVI.—MILD STEEL IN TENSION AND COMPRESSION.

Comparison of Tensile and Compressive Results with Results of Tests on Short Columns of Round and Square Bars from ¾ in. to 2½ in. in diameter, all rolled from one blow of Bessemer Steel. Elongations measured on a length equal to ten diameters, by means of the Marshall Extensometer shown in Fig. 271. (From Marshall's Experiments, *Trans. Am. Soc. C. E.*, vol. XVII, Tables I and II.)

Size of Specimen	Elastic Limit in Pounds per Square Inch		Ultimate Strength in Pounds per Square Inch		Elongation.		Reduction of Area, Percentage.
	in Tension.	in Compr. for $\frac{l}{d} = 2$.	in Tension.	in Compr. for $\frac{l}{d} = 12$.	Length of Specimen.	Percentage of Elongation.	
3/4	45,181	45,000	68,711	44,970	8	26.4	45.3
1	43,880	45,355	68,240	43,540	10	25.6	39.3
1¼	40,903	42,880	67,506	40,455	12	26.4	43.0
1½	39,795	42,015	66,598	40,150	15	25.4	39.3
1¾	39,105	41,225	66,366	39,700	18	24.3	33.8
2	38,207	39,176	65,663	40,300	20	23.9	27.8
2¼	37,655	36,542	65,460	38,030	22	18.7	17.2
2½	36,100	36,840		35,650	25	10.2	
Means	40,103	41,129	66,935	40,350		21.9	35.0

FIG. 416.—Showing the Manner in which Rectangular Steel Test-specimens reduce in Cross-section. (*Engr. News*, vol. XXXIII. p. 272.)

on the freedom with which the metal can flow laterally. Thus in Fig. 417 we have a column compressed over its full cross-section with freedom to flow laterally in every direction. This is the usual condition under which the elastic limit in compression is found.

In Fig. 418 the specimen is compressed uniformly over a portion only of its surface, and when the elastic limit is exceeded the metal finds escape by flowing laterally against the resistance of a ring of unstressed metal. This is a condition of restricted flow, and evidently the elastic limit now is much higher than before.

In Fig. 419 only the metal towards the centre of the compressed surface is constrained to flow under the direct stress, but in attempting to move laterally it is held by a ring of metal which is confined and compressed vertically, though inside its elastic limit. To find an escape the metal at the

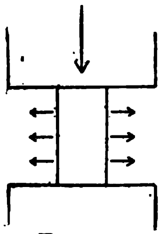

Fig. 417.
Free Flow.

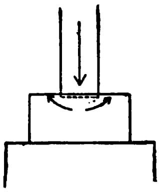

Fig. 418.
Restricted Flow.

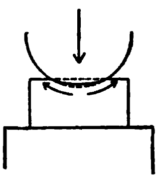

Fig. 419.
Confined Flow.

centre must force its way against a much wider ring of metal than in the second case, and hence the elastic limit now is very much higher than when pressed by a flat disk.

The elastic limit in compression, therefore, is a meaningless expression unless the conditions of lateral flow are also stated.

364. The Author's Tests of Areas of Contact between Car-wheels and Rails.—In Figs. 420 and 421 are shown a series of actual areas of contact obtained by pressing sections of a cast-iron car-wheel and of a locomotive steel driving-wheel upon the cylindrical top surface of a steel rail. This was done in a testing-machine in such a way that there was no rocking motion and the area of contact was clearly distinguished.*

The areas of these surfaces of contact were determined by a planimeter, and these are plotted to their corresponding loads in Fig. 422. It will be seen that these plot in nearly a straight line through the origin. If such a law be assumed, it follows—

1. *That the area of contact increases directly with the load.*

2. *That the mean intensity of pressure is a constant for all loads.*

3. *That in these experiments this mean intensity of compressive stress, for all loads, was about* 82,000 *lbs. per square inch.*

4. Since the maximum deformation (at the centres of these areas) is twice the average deformation (assuming the volumetric deformation to be that of a segment of a paraboloid of revolution), then *the maximum compressive-stress intensity for all loads is about* 164,000 *lbs. per square inch.*

5. Since no measurable permanent set was produced by any of these loads on either wheels or rail, it follows that *the "apparent elastic limits" of the materials had not been reached for this condition of contact, although the ordinary elastic limit of the rail material, for a free flow as in Fig.* 417, *was about* 50,000 *lbs. per square inch.*

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* See a full account of these tests, showing other areas of contact, in *Trans. Am. Soc. Civ. Engrs.*, vol. XXXII. p. 270, 1894.

IMPRESSIONS ON 75-LB. STEEL RAIL. TOP RADIUS, 14 IN. FULL SIZE.

Direction along the Rail.

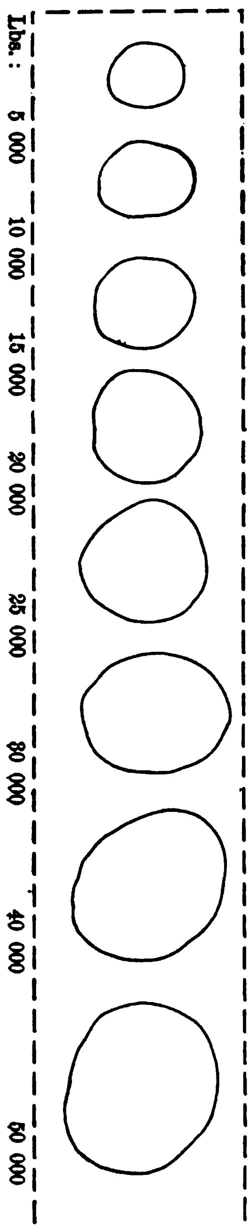


FIG. 420.—Steel Driver, 44 in diam. Flat tread

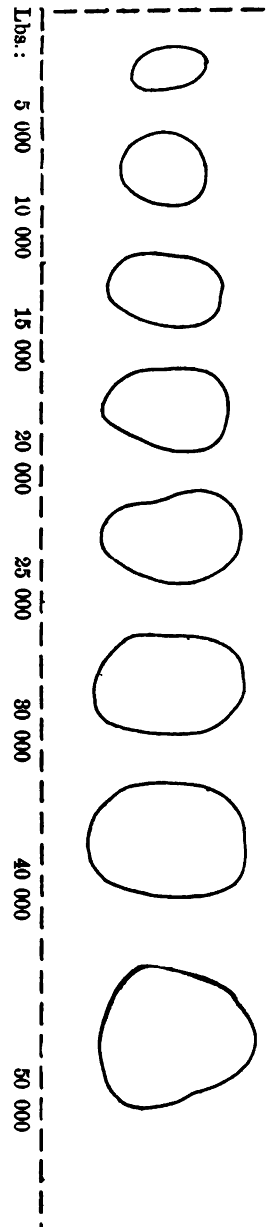


FIG. 421.—Chilled Wheel, 33 in. diam. New.

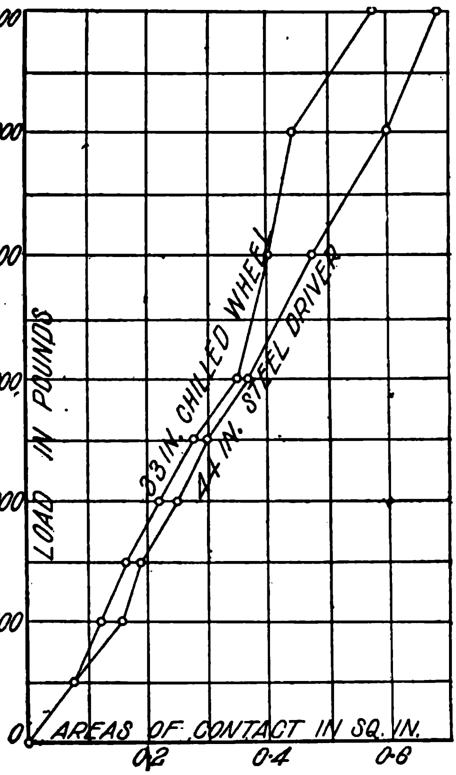


FIG. 422.—Showing the Relation between the Total Load and the Area of Contact between Wheels and Rails. (Johnson, in *Trans. Am. Soc. C. E.*, vol. XXXII.)

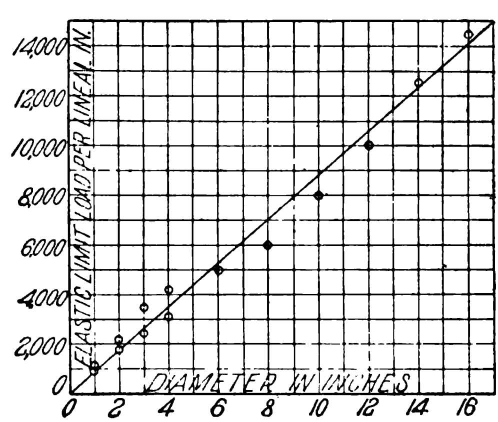


FIG. 423.—The Elastic-limit Loads per Lineal Inch of Rollers of Various Diameters. (Crandall and Marston, in *Trans. Am. Soc. Civ. Engrs.*, vol. XXXII. p. 120 (1894).)

These are important conclusions, and should be supplemented and supported by further observations of this character.

In Fig. 423 are shown the results of tests made by Profs. Crandall and Marston to find the elastic-limit loads on steel cylinders resting on or between steel plates. These results show that the elastic loads vary directly with the diameters, these loads per lineal inch of rollers, for mild structural steel, being

$$p = 880d, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where p = elastic-limit load in pounds per lineal inch, and d = diameter of roller in inches.

365. The Moduli of Elasticity in Tension and Compression for various sizes and qualities of steel and wrought iron are given in Table XXVII,

TABLE XXVII.—COMPARISON OF MODULI OF ELASTICITY IN TENSION AND COMPRESSION.*

All results given in one-thousand-pound units, identical material.

STEEL.—Tensile Strength less than 100,000 lbs. per Square Inch.					SPRING-STEEL.—Tensile Strength 144,000 Pounds per Square Inch.				
Size of Bar.	Tension. E_1 First Loading.	Tension. E_2 Second Loading.	Compression. E_1 First Loading.	Compression. E_2 Second Loading.	Size of Bar.	Tension. E_1 First Loading.	Tension. E_2 Second Loading.	Compression. E_1 First Loading.	Compression. E_2 Second Loading.
1 rd.	30,190	**34,420**	29,450	**29,740**	1 rd.	29,480	**29,760**	28,880	**29,300**
$\frac{8}{10}$ sq.	29,850	**29,850**	28,070	**29,010**	1 rd.	29,390	**29,580**	28,880	**29,200**
1 rd.	29,280	**29,500**	28,780	**29,420**	$\frac{8}{10}$ sq.	28,880	**29,420**	29,090	**29,220**
$\frac{3}{4}$ rd.	29,830	**29,150**	28,580	**29,420**	$\frac{8}{10}$ sq.	29,200	**29,200**	29,090	**29,350**
$\frac{8}{10}$ sq.	29,420	**29,640**	28,380	**28,670**	mean	29,237	**29,490**	28,985	**29,267**
1 rd.	29,550	**29,630**	28,680	**28,830**					
1 rd.	29,240	**29,960**	30,070	**30,490**					
1 rd.	29,400	**30,420**	28,980	**29,790**					
1 rd.	30,000	**30,370**	29,260	**29,810**					
mean	29,529	**30,371**	28,884	**29,464**					

WROUGHT IRON.

Size of Bar.	Tension. E_1 First Loading.	Tension. E_2 Second Loading.	Compression. E_1 First Loading.	Compression. E_2 Second Loading.	Size of Bar.	Tension. E_1 First Loading.	Tension. E_2 Second Loading.	Compression. E_1 First Loading.	Compression. E_2 Second Loading.
$\frac{3}{4}$ rd.	26,800	**27,500**	25,840	**26,160**	1 sq.	28,290	**28,290**	27,100	**27,990**
$\frac{3}{4}$ rd.	26,980	**27,410**	25,920	**26,240**	1 rd.	27,590	**28,570**	27,250	**28,570**
1 rd.	...	**26,700**	25,670	**26,440**	1 rd.	28,290	**28,480**	27,430	**28,570**
1 rd.	27,540	**27,540**	26,020	**26,350**	1 rd.	26,580	**28,480**	26,500	**28,180**
$\frac{3}{4}$ sq.	28,990	....	27,420	....	$\frac{7}{8}$ rd.	30,190	**30,190**	29,520	**29,910**
$\frac{3}{4}$ sq.	27,790	**29,180**	25,650	**27,790**	mean	27,894	**28,203**	26,734	**27,590**
1 sq.	27,800	**27,900**	26,490	**27,300**					

* From experiments by Charles A. Marshall, M. Am. Soc. C. E., reported in *Trans. Am. Soc. Civ. Engrs.*, vol. XVII. pp. 62–3.

these results also being from Marshall's experiments. They show the relation between the modulus of elasticity as obtained from the first and from the second loading, the first loading not having been carried beyond the elastic limit. As the modulus from the second loading is always a little larger than that obtained from the first loading, it shows that *on the first loading there is always a small permanent set, and that moduli of elasticity should be observed only after a load has been imposed and removed.* The

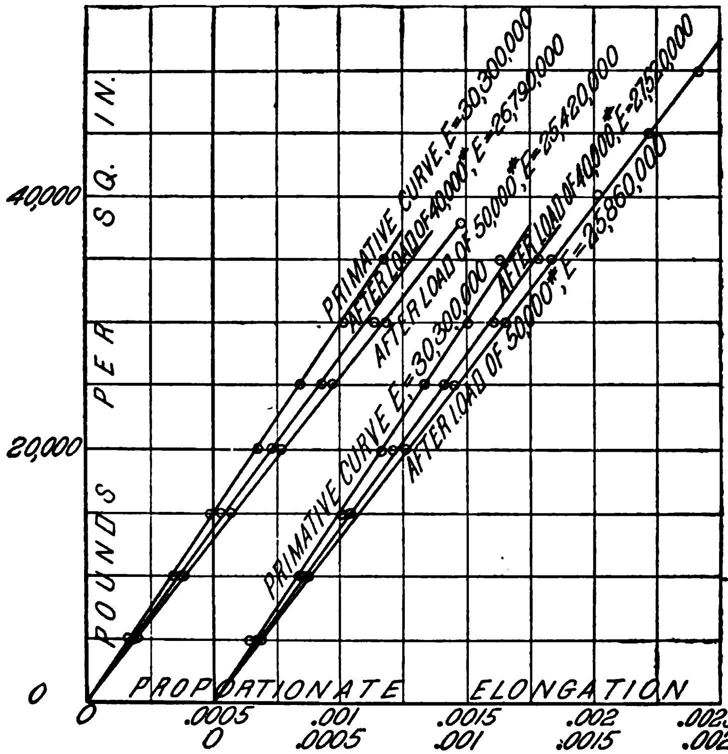


FIG. 424.—Showing Variations in the Modulus of Elasticity of Steel Eye-bars, 66,000 lbs. T. S., after the Elastic Limit has been Passed. (*Wat. Ars. Rep.* 1883.)

failure to do this may explain some of the low values of this modulus which are often given.

If the specimen be stretched much beyond its elastic limit, however, the elastic limit is lowered after each such higher loading, as indicated in Fig. 424.

366. Modulus of Elasticity Independent of the Other Mechanical Properties.—In Table XXVIII are given the average values of the moduli of elasticity from 262 determinations on steels of five degrees of hardness. The mean values for these five classes do not in any case differ from the mean of all by more than *six tenths of one per cent.* As the mean of all is

TABLE XXVIII.—MODULI OF ELASTICITY OF STEEL ON FIRST LOADINGS, WITH VARYING PERCENTAGES OF CARBON, ONE SPECIMEN FROM EACH HEAT.*

Number of Heats and Tests.	Average Percentage of Carbon.	Moduli of Elasticity *E*, in Pounds per Square Inch. Lowest Value.	Highest Value.	Average Value.	Kind of Steel.
33	9	28,750,000	31,540,000	29,924,000	Bessemer
8	11	29,210,000	30,670,000	30,020,000	Open-hearth
107	24	28,310,000	31,180,000	29,996,000	" "
89	34	28,140,000	30,910,000	29,672,000	Bessemer
25	72	28,680,000	30,860,000	29,919,000	Open-hearth
			Weighted mean value =	29,866,000	

* From Marshall's Experiments, *Trans. Am. Soc. C. E.*, vol. XVII. p. 64.

TABLE XXIX.—TENSILE TESTS ON ROUND STEEL RODS FROM 1 TO 3 INCHES IN DIAMETER, ANNEALED AND UNANNEALED.

Each recorded result is the mean of three tests. All the results in one horizontal line are for tests on material cut from the same three bars.†

Size.	Elastic Limit‡ in Pounds per Square Inch.			Ultimate Strength in Pounds per Square Inch.			Ratio of Elastic Limit to Ultimate Strength.		
	Unannealed.		Annealed.	Unannealed.		Annealed.	Unannealed.		Annealed.
Diam. in in.	Rods 100 in. long.	Rods 10 in. long.	Rods 10 in. long.	Rods 100 in. long.	Rods 10 in. long.	Rods 10 in. long.	Rods 100 in. long.	Rods 10 in. long.	Rods 10 in. long.
1	43,330	46,970	45,130	63,810	66,050	62,010	67.8	71.1	72.7
1½	42,400	43,300	42,170	62,507	65,020	62,460	67.8	66.5	67.5
2	36,520	39,570	36,270	61,320	61,420	58,490	59.5	64.4	62.0
2½	34,130	37,230	34,300	58,950	60,300	56,790	57.8	61.7	60.3
3	35,700	36,530	33,500	58,550	59,830	57,370	60.9	61.0	58.3

Size.	Percentage of Elongation.			Percentage of Reduction.			Modulus of Elasticity of the 100-inch Bars Unannealed in Pounds per Square Inch, First Loading.
	Unannealed.		Annealed.	Unannealed.		Annealed.	
Diameter in inches.	Rods 100 in. long	Rods 10 in. long.	Rods 10 in. long.	Rods 100 in. long.	Rods 10 in. long.	Rods 10 in. long.	
1	19.21	25.6	22.1	60.2	61.1	65.7	27,300,000
1½	21.42	26.9	24.9	55.3	55.0	58.4	21,100,000
2	25.62	30.9	30.4	58.8	59.4	62.5	30,000,000
2½	23.50	31.4	32.5	56.9	54.9	62.7	30,600,000
3	17.34	30.6	33.9	54.6	49.8	61.2	28,400,000

† From *Kirkaldy's Report*, 1891, reports M and HH.
‡ This is the true elastic limit on the first loading; it was about 5 per cent below the yield-point, or the "apparent elastic limit."

TABLE XXX.—COMPARISON OF TENSION AND COMPRESSION TESTS ON ANNEALED AND UNANNEALED STEEL BARS OF IDENTICAL MATERIAL WHICH HAD BEEN STRESSED BEYOND ITS ELASTIC LIMIT.*

Tension Tests.										Compression Tests.					
Marks.	How Taken.	Condition.	Diameter.	Sectional Area.	Elastic Limit per Sq. Inch.	Ultimate Strength per Sq. In.	Elongation in 5 In.	Contraction of Area.	Modulus of Elasticity.	Marks.	Diameter.	Sectional Area.	Elastic Limit per Sq. Inch.	Ultimate Strength per Sq. In.	Modulus of Elasticity.
			in.	sq. in.	lbs.	lbs.	per cent	per cent	lbs. per sq. in.		in.	sq. in.	lbs.	lbs.	lbs. per sq. in.
1	Crosswise	Unannealed	.564	.25	40,000	66,320	12.0	24.6	30,300,000	3	.926	.673	51,000	58,100	34,300,000
2	"	Annealed	.564	.25	40,000	63,160	22.0	33.5	31,200,000	4	.942	.697	43,000	43,000	33,000,000
5	Diagonally	Unannealed	.564	.25	40,000	66,760	16.0	59.8	30,200,000	7	.926	.673	47,000	54,380	34,300,000
6	"	Annealed	.564	.25	42,000	62,840	27.5	61.5	30,700,000	8	.935	.687	40,000	43,000	31,800,000
9	Lengthwise	Unannealed	.564	.25	63,000	73,000	18.0	61.5	31,200,000	11	.915	.658	35,000	48,940	34,900,000
10	"	Annealed	.564	.25	47,000	64,000	26.0	61.5	30,700,000	12	.933	.684	42,000	45,000	32,400,000

* These specimens were cut from an eye bar which had been stressed to 54,350 lbs. per square inch, 3 yrs. and 8 mos. before these tests were made. The original elastic limit was 34,400 lbs. per square inch. See Fig. 425, showing original position of specimens. (From *Report of Watertown U. S. Arsenal Tests*, 1890, p. 731.)

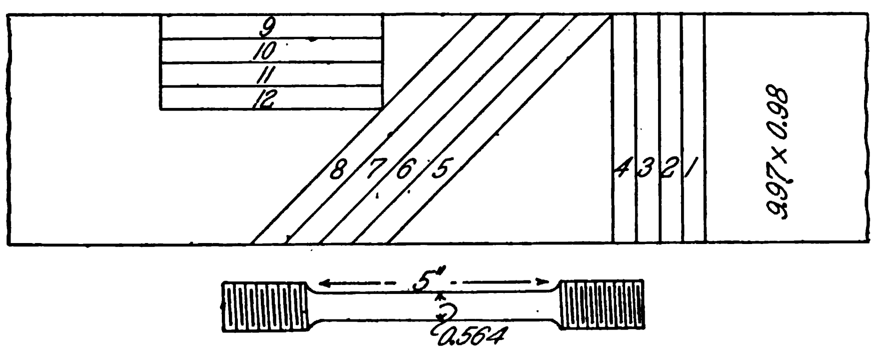


FIG. 425.—Showing Location of Test-specimens cut from an Overstrained Steel Bar after a rest of 3 yrs. and 8 mos. (*Wat. Ars. Rep.* 1890.)

29,866,000, and this from first loadings, it will not be appreciably in error to call the modulus of elasticity of steel 30,000,000. (See also Figs. 398 to 403, where tensile stress indicated for an elongation of 0.001 is uniformly about 30,000 lbs. per square inch, thus giving a modulus of elasticity of 30,000,000.)

367. The Effect of Annealing on Steel Before and After Overstraining.— In Table XXIX are given results of annealing steel bars which have never been stressed or worked cold. In Table XXX are given the results of tests

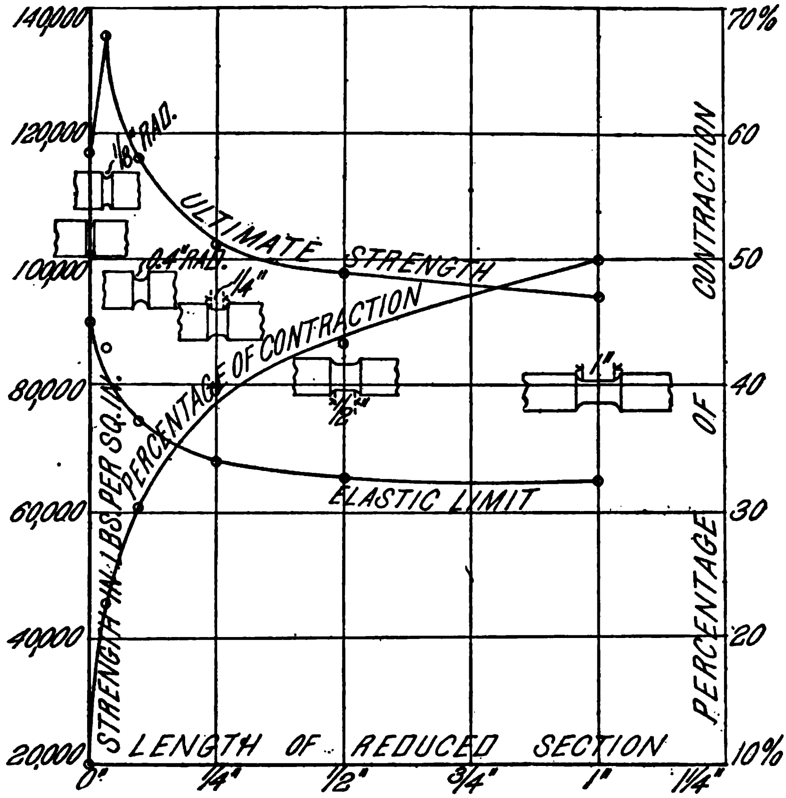


FIG. 426.—Effect of Length of the Reduced Section on the Strength and Ductility of Steel. James E. Howard (in charge of Tests at the Watertown Arsenal) before Inter. Eng. Cong. 1893. Section of *Nav. Eng. and Marine Arch.*, vol. II, J. Wiley & Sons, New York.

on overstrained steel which was afterwards annealed. It will be observed that, while it has little effect on soft steel in the normal state, the annealing largely restores the original qualities to overstrained soft steel, though it does not fully do so. The high moduli of elasticity in compression given in Table XXX for both the annealed and the unannealed specimens should be accepted with caution as being probably erroneous.

368 The Effect of Varying the Length of the Reduced Section.*—"The following results of the tests of six specimens from the same 1¼-in. steel bar illustrate the apparent elevation of elastic limit and the changes in other properties due to changes in the lengths of stems which were turned down in each specimen to 0.798 in. diameter. (See also Fig. 426.)

Description of Stem.	Elastic Limit, Pounds per Square Inch.	Tensile Strength, Pounds per Square Inch.	Contraction of Area, Per Cent.
1.00" long	64,900	94,400	49.0
.50 "	65,320	97,800	43.4
.25 "	68,000	102,420	39.6
Semicircular groove, 0.4" radius	75,000	116,380	31 6
Semicircular groove, 1/8" radius	86,000, about	134,960	23.0
V-shaped groove	90,000, about	117,000	Indeterminate

"These tests show the progressive elevation of the elastic limit as the stems of the specimens were shortened, and the corresponding effect upon the tensile strength. The contraction of area, of course, diminishes as the other two features increase in value.

"The lower tensile strength of the specimen having the V-shaped groove was probably due to the excessive concentration of stress at the bottom of the groove from inability to elongate or contract, fracturing the metal more in detail than happened to the other specimens."

In Fig. 427 are shown the results of similar tests made by M. Duguet on hard steel and by M. Barba on soft steel bars. In both of these sets of experiments the very short reduced sections have a greatly increased breaking strength.

In this connection it must be remembered that in ductile metals, where the reduced section has appreciable length, there is a great reduction of area, so that the stress per square inch at rupture on the actual section at that time is about twice the tensile strength as computed on the original cross-section. In the very short or grooved reduced sections, however, the material has no opportunity to reduce in area, and hence the actual rupturing stress is developed over the full original area. In the case of the sharp V-shaped groove the material is likely to tear apart by failing first at the outer edges. In other words, the stress is not uniformly distributed over the cross section.

In Fig 428 are plotted the results of tests of the same grade of steel (54,000 lbs tensile strength), when tested in the standard form, with parallel sides, and when grooved as has long been required for the U. S. Marine Service The effect of the groove is to raise the tensile strength from 7000

* Quoted paragraphs and table taken from a paper by James E. Howard read before the *World's Engineering Congress*, 1893.

lbs. per square inch on the ⅜ inch plate to over 12,000 lbs. per square inch on the 1-inch plate. The grooved specimen, furthermore, gives little or no

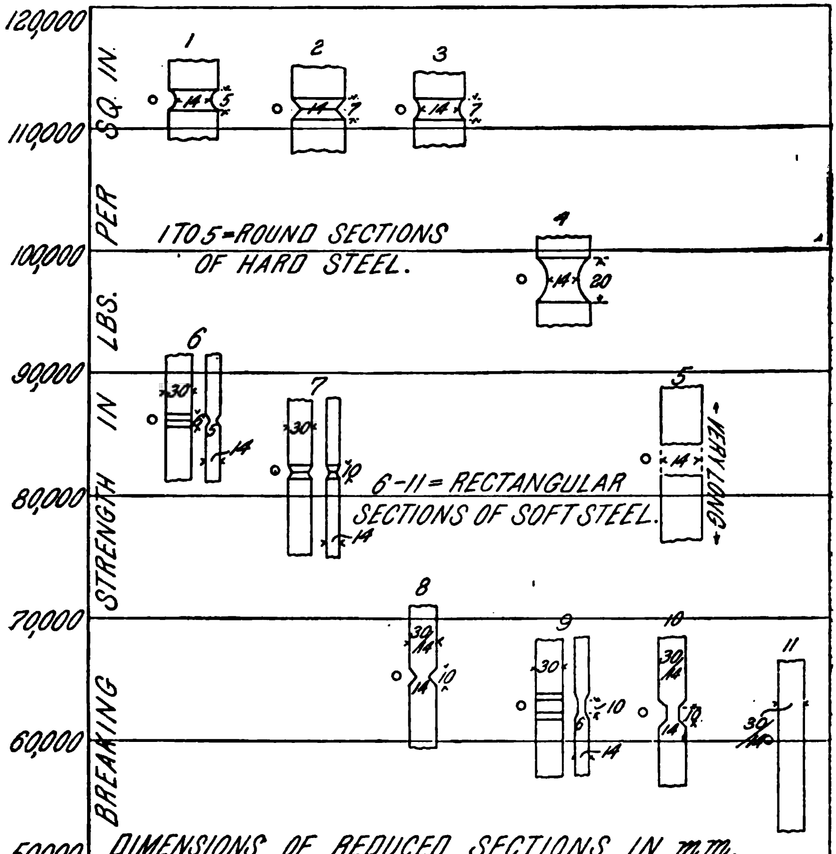


FIG. 427.—Showing the Effect of the Form of the Reduced Section on the Tensile Strength of Two Kinds of Steel. (*Fr. Com. Rep.*, vol. III, p. 40.)

indication of the elastic limit, and no indication of the percentage of elongation. The requirement of grooved specimens on this service will probably soon be abandoned.

369. Nickel-steel,* being an alloy of mild steel with about 3¼ % of nickel, has a very high elastic limit and ultimate strength, combined with great ductility, as shown in Fig. 429. This alloy is doubtless destined to play a

* First made by Marbeau in 1885, and used for armor-plate in 1890. The price of nickel steel was 40 to 45 cents a pound in 1894. For a complete study of the influence of nickel on pure iron, in all proportions, see *Berlin Testing Laboratory Communications*, 1896, vol. IV. p. 222.

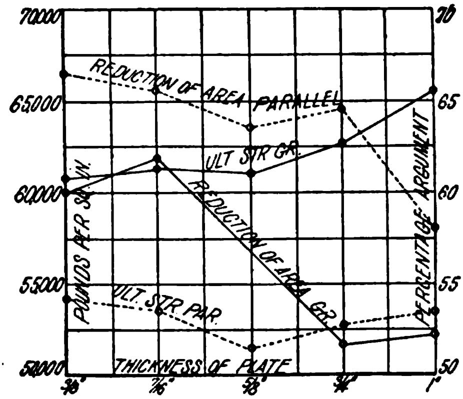


Fig. 428.—Showing Relative Results from Grooved and Parallel-sided Specimens (Campbell's *Structural Steel*, p. 228.)

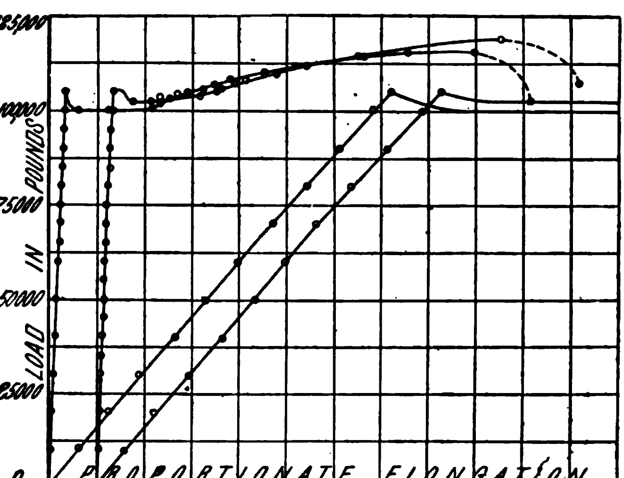


Fig. 429.—Tension Tests on Nickel-steel. (*Wat. Ars. Rep.* 1894, pp. 199 and 200.)

leading part wherever great elastic strength and a reasonable ductility are required. It would seem to be especially fitted for bicycle tubing and spokes, aerial experimentation, the reciprocating parts of locomotive engines, motor carriages, etc., as well as for armor-plates.

370. The Mechanical Properties of Steel as Affected by Forging and Rolling.—In Fig. 430 is shown the cross-section of a steel shaft 16 inches in diameter (which broke soon after being put in service) from which eight test-

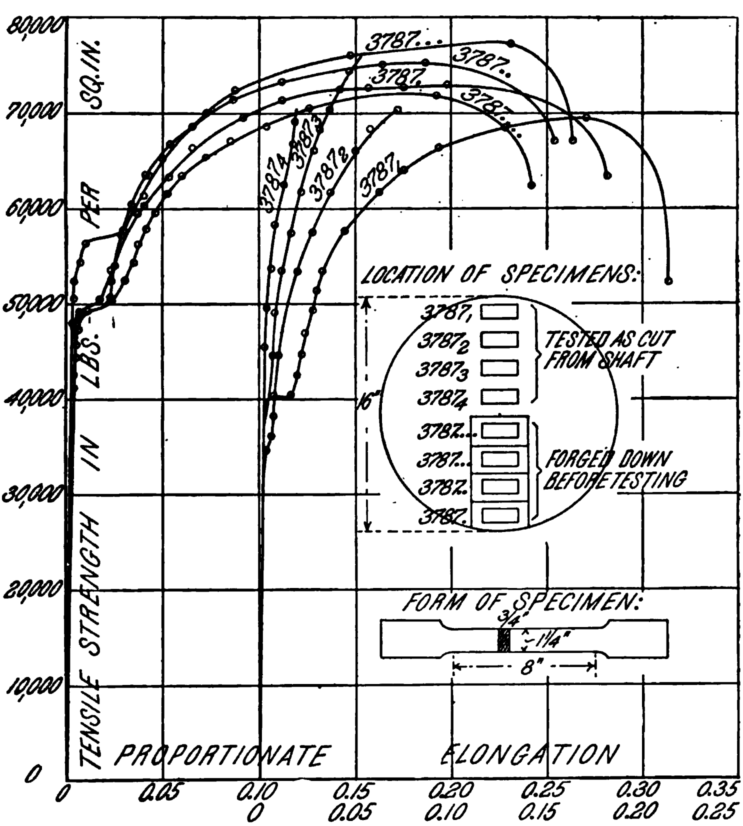


FIG. 430.—Showing the Varying Character in the Material in Different Parts of the Cross-section of a Large Steel Shaft when forged under a Ten-ton Hammer. (*Wat. Ars. Rep.*, 1885.)

specimens were cut, lying symmetrically in a diametral section as shown. Four of these were tested as cut from the shaft. The other four were forged down after cutting out. The plotted results show—

1. The elongation of the unforged specimens varied from 21 per cent in the

specimen taken from near the surface of the shaft to 2 per cent in the specimen coming from near the centre. In the forged specimens, however, taken from the opposite side of the disk, the elongation varied from 28 per cent near the surface of the shaft to 24 per cent near the centre, thus showing that the material was identical throughout when it had been similarly worked. In other words, the material near the centre of the shaft was in its primitive condition when first cast, while that near the surface was that of well-rolled steel. This shows the necessity of forging large shafts under enormously heavy hammers, or, better, the necessity of using only hollow-forged shafts for such service.

Steel car-axles are now rolled and then finished by drawing through a die. In Fig. 431 are shown the effects of a series of 16 blows upon a steel car-axle, $3\frac{3}{8}$ in. in diameter, turned over after each second blow, of a drop weighing 1640 pounds and falling from 18.8 to 28 ft. The deflections from each blow varied from $8\frac{1}{2}$ to 13 in., but the axle remained unbroken after this severe treatment.

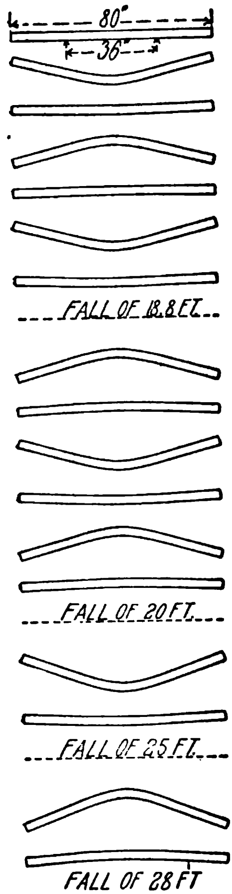


FIG. 431.—Showing the Successive Forms of a $3\frac{3}{8}$-in. diam. Steel Car-axle when tested by Impact without Rupture. Wt. of drop = 1640 lbs. (Original MS.)

371. Steel-welded Tubes.—In Tables VII and VIII, pp. 130 and 131, it was shown that steel may be welded as securely as wrought iron, but that the temperature at which this material welds perfectly lies within comparatively narrow limits. If heated above the upper limit the steel melts and oxidizes, and it is then said to have been burned. If not heated up to the lower limit an imperfect weld is formed. To effect a good weld in a common blacksmith's forge therefore requires great skill and care. On the other hand, where steel plates used for tube-making are uniformly heated in a furnace in which the temperature can be maintained constant and of a given degree, these may then be welded perfectly, especially if this be done by machinery. The evidence of such perfect welds is furnished in Figs. 432 and 433, where welded steel tubes are shown to have been subjected to the most severe abuse, by cold crushing and twisting, and without any failure appearing in any of the welded joints.

372. Wrought-iron and Steel I Beams and Plate Girders.—When rolled into I beams, or when plates and angles are riveted together to form a plate girder, the true elastic limit of the beams and girders is below that of the specimen test-pieces cut from the webs and flanges. The ultimate strength of the wrought-iron beams and girders is higher than that of the specimens,

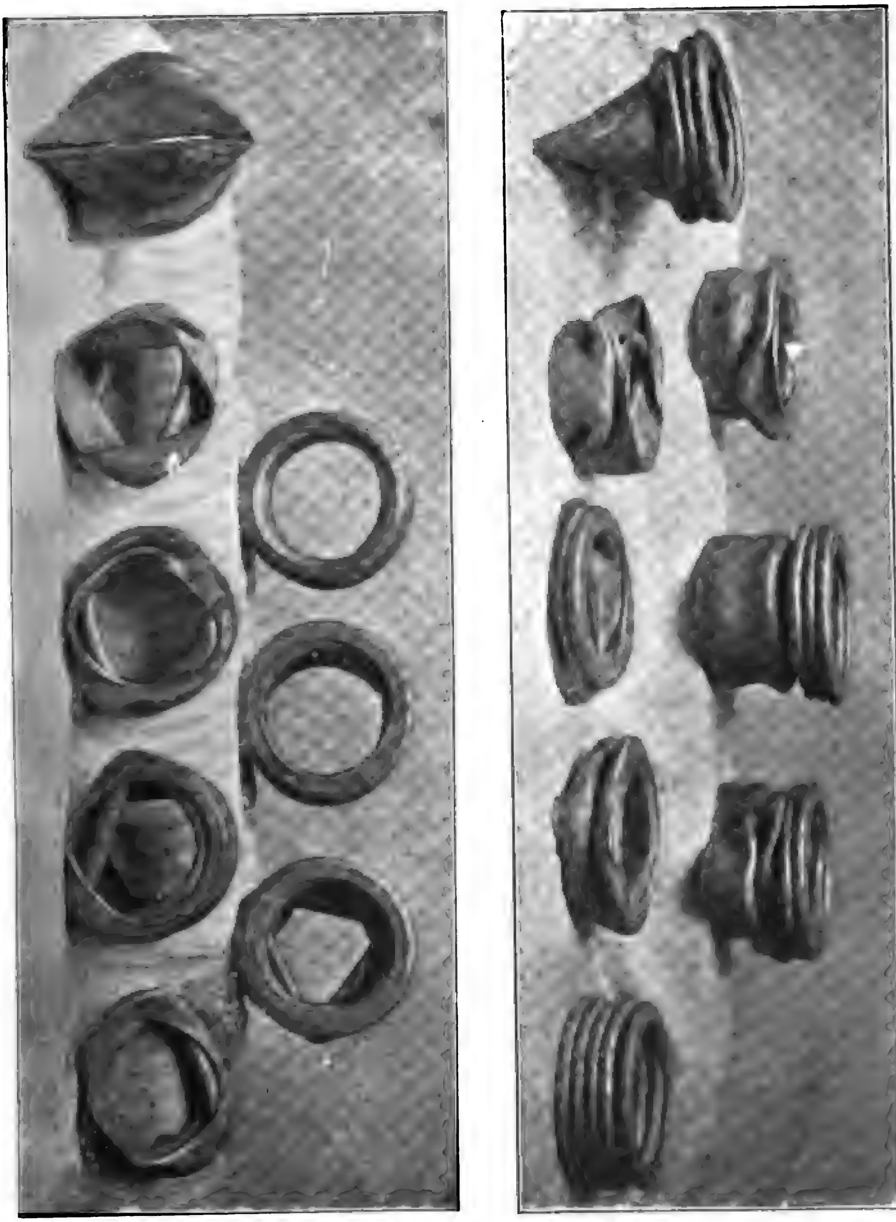


Fig. 432.—Examples of the Cold Deformation of Welded Steel Tubing, made by the National Tube Works at McKeesport, Pa.
(From the *Iron Age*, Sept. 17, 1896.)

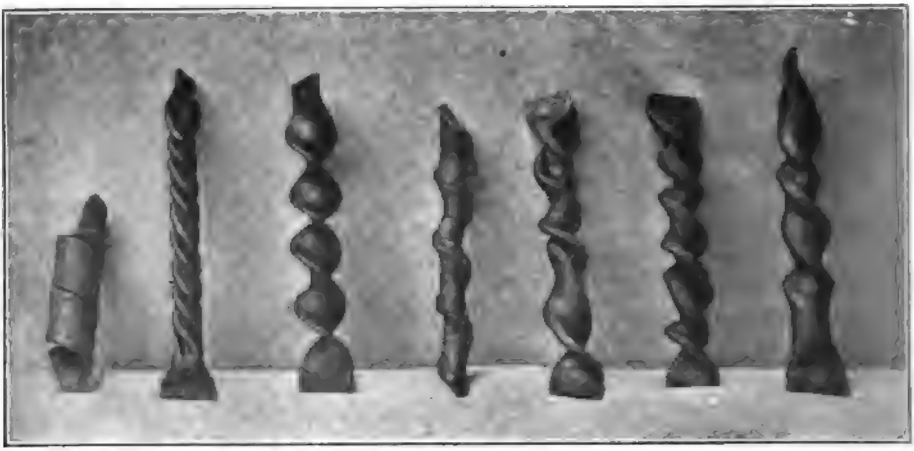


FIG. 433.—Examples of Welded Steel Tubes twisted Cold, made by the National Tube Works. (From *The Iron Age*, Sept. 17, 1896.)

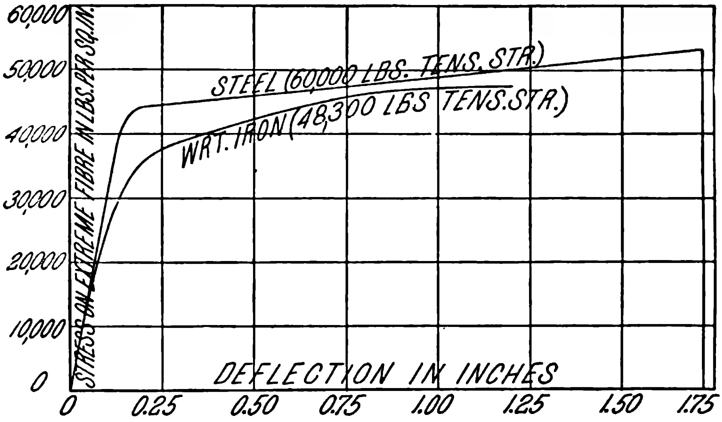


FIG. 434.—Bending Tests on Steel and Wrought-iron I Beams, 7.6 in. high, on 60-in. span. (Tetmajer, vol. III, Pl. IV.)

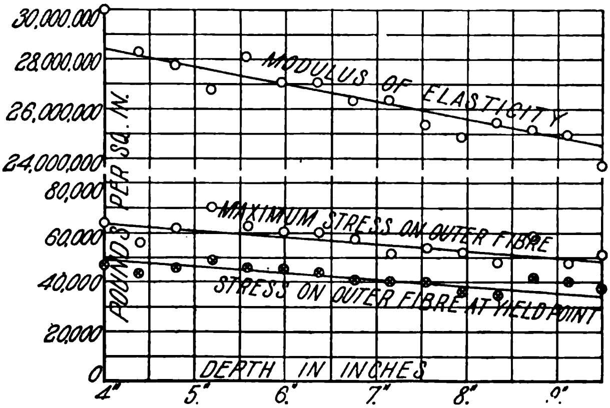


FIG. 435.—Showing Variation in Moduli of Strength and Stiffness of Steel I Beams with Varying Depth. (Tetmajer's *Communications*, vol. III, Pl. V.)

but with the steel beams and girders the reverse is the case. All these relations are shown by the following tables, which are the results of a series of very careful tests made by Professor Tetmajer. The moduli of elasticity of the rolled or riveted forms are not appreciably lower than those of the materials of which they are composed; but this modulus and also the moduli of strength seem to decrease with increasing heights of beam, as shown in Fig. 435.

TABLE XXXI.—ELASTIC LIMIT AND ULTIMATE STRENGTH OF WROUGHT-IRON I BEAMS AS COMPARED WITH RESULTS OF TESTS OF SPECIMENS CUT FROM THE WEBS AND FLANGES OF THE SAME.

(Each result is the mean of two tests. From Prof. von Tetmajer's *Communications*, vol. IV.)

Depth of Beam in Inches.	Elastic Limit in Pounds per Square Inch.			Ultimate Strength in Pounds per Square Inch.			Per Cent of Elongation.		Modulus of Elasticity of I Beams in Pounds per Square Inch.
	Web Specimen.	Flange Specimen.	Beam, Extreme Fibre.	Web Specimen.	Flange Specimen.	Beam, Extreme Fibre.	Web Specimen.	Flange Specimen.	
4	40,520	42,230	35,120	54,740	57,300	62,850	10.4	19.1	28,600,000
6	38,680	36,400	33,270	51,330	53,750	56,450	7.8	25.5	28,300,000
8	35,690	34,840	33,840	50,900	51,900	53,890	11.7	18.5	28,300,000
10	36,120	34,410	31,000	48,490	51,330	51,620	13.1	15 8	27,500,000
12	31,000	32,280	31,570	44,930	52,610	58,180	9.1	19.7	26,500,000
14	33,840	33,700	27,370	51,470	53,460	53,890	17.9	20.5	27,700,000
16	33,130	31,140	29,720	50,480	50,190	52,470	11.9	22.9	27,600,000
Means	35,572	35,000	31,770	50,334	52,934	53,478	11.7	20.2	27,800,000

TABLE XXXII.—ELASTIC AND ULTIMATE STRENGTH OF WROUGHT-IRON PLATE GIRDERS COMPOSED OF A SOLID WEB, FOUR ANGLES, AND TWO COVER-PLATES, AS COMPARED WITH THE TENSILE STRENGTH OF THE PARTS COMPOSING THEM.

(Each result is the mean of tests on two beams or on four tension specimens. From von Tetmajer's *Communications*, vol. IV.)

Test-specimen.	Elastic Limit in Pounds per Square Inch.	Ultimate Strength in Pounds per Square Inch.	Percentage of Elongation in 8 Inches.	Percentage of Reduction of Area.	Modulus of Elasticity of Girders in Pounds per Square Inch.
Web-plate lengthwise..........	40,950	53,320	14.1	16.2	....
" crosswise.........	36,120	37,400	0.5	0.4	....
Cover-plates lengthwise........	35,120	51,040	13.9	17.0	....
Angles lengthwise............	34,690	46,350	8.4	14.6	....
Plate girders 16 in. high........	25,590	52,330	....	....	25,990,000
" " 20 in. high........	29,860	52,040	....	....	25,250,000
" " 24 in. high........	......	50,620	....	....	26,430,000
" " 28 in. high.	27,580	47,210	....	....	26,220,000
Mean of the parts.............	38,720	47,050	9.2	11.8	....
Mean of the girders............	27,680	50,550	....	....	25,970,000

TABLE XXXIII.—ELASTIC AND ULTIMATE STRENGTH OF MILD-STEEL PLATE GIRDERS COMPOSED OF A SOLID WEB, FOUR ANGLES, AND TWO COVER-PLATES, AS COMPARED WITH THE TENSILE STRENGTH OF THE PARTS COMPOSING THEM.

(Each result is the mean of tests on two beams or on four tension specimens. From von Tetmajer's *Communications*, vol. IV.)

Test-specimen.	Elastic Limit in Pounds per Square Inch.	Ultimate Strength in Pounds per Square Inch.	Percentage of Elongation in 8 Inches.	Percentage of Reduction of Area.	Modulus of Elasticity of Girders in Pounds per Square Inch.
Web-plate lengthwise	52,380	64,560	24.1	59.5	....
" crosswise	53,040	65,980	20.0	46.2	....
Cover-plates lengthwise	51,190	64,840	25.0	56.2	....
Angles lengthwise	39,960	53,750	31.0	66.1	....
Plate girders 16 in. high	32,560	55,310	....	....	27,700,000
" " 20 in. high	35,690	54,320	....	....	28,310,000
" " 24 in. high	33,700	55,080	....	....	28,510,000
" " 28 in. high	32,280	53,750	....	....	28,140,000
Mean of the parts	49,130	62,280	25.0	57.0	....
Mean of the girders	33,550	54,600	....	....	28,165,000

373. The Effect on Mild Steel of Stressing it Beyond its Elastic Limit.—Both wrought iron and rolled steel, in their normal state, have "apparent elastic limits" in tension and in compression numerically about equal. If this material be stressed much beyond these limits, however, in either direction, its elastic limit in this direction is numerically raised to about the limit of its greatest stress, while the elastic limit in the opposite direction is greatly lowered or even reduced to zero. This is well shown in Figs. 436 and 437. Thus in Fig. 436 a steel specimen was stressed several times in tension between zero and 45,000 lbs. per square inch, when it was put in compression to 50,000 lbs. per square inch, at which load it passed its elastic limit and began to flow. After compressing it 0.007 of its length the load was removed and simultaneous readings of load and deformation taken while the load was coming off, as shown in the figure. The specimen was then worked between 10,000 lbs. tensile and a like compressive stress, then between zero and 20,000 lbs. tension, and zero and 40,000 lbs. tension, and finally between zero and 50,000 lbs. per square inch tension, when it had lengthened 0.004 beyond its original length as shown in Fig. 436. It was then put in compression under a load of 50,000 lbs. per square inch, which compressed it 0.004 below its original length, while a subsequent tensile stress of 50,000 lbs. brought it nearly back to its previous deformed length under this tensile stress. The diagram shows the following remarkable facts:

1. *A permanent deformation of one half of one per cent in either tension or compression entirely destroys the perfect elasticity of the material under the opposite kind of stress.*

This is shown by the fact that the stress-diagram becomes a curved line under all stresses of one kind after having been given a small permanent set in the opposite direction. Hence we have:

2. *The elastic field which is symmetrically placed about the line of zero*

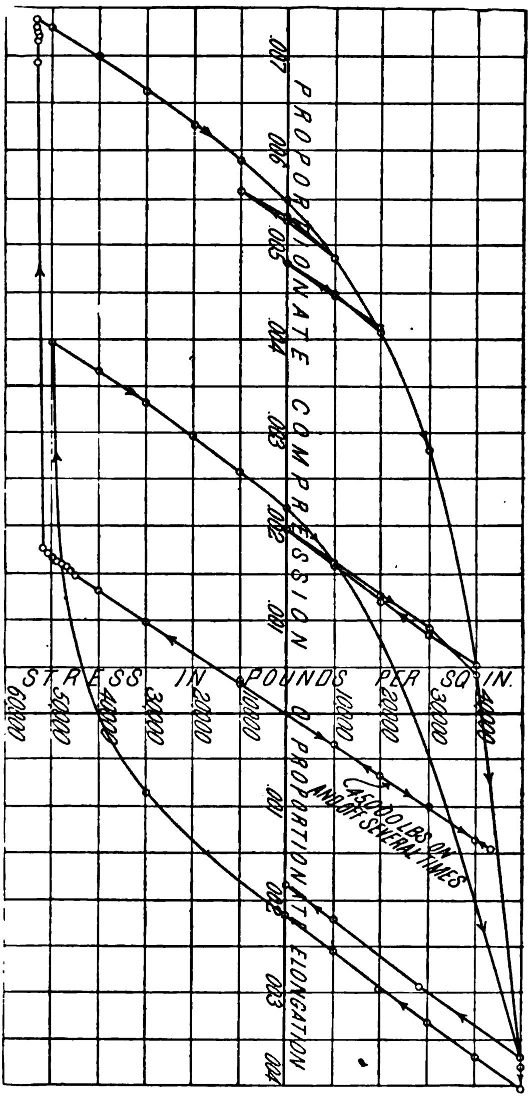


FIG. 436.—Alternate Tensile and Compressive Deformations, beyond the Elastic Limits, of a Steel Bar. (Wat. Ars. Rep. 1889.)

stress in the normal specimen becomes wholly limited to that side of this axis on which the stress has exceeded the elastic limit.

A similar set of experiments, plotted in Fig. 437, was followed by the annealing of the bar, after which it showed again its normal elastic limits in

both tension and compression, which were in turn again destroyed by deforming the annealed bar 0.003 beyond its elastic limits, as before. Hence we may say:

3. *Annealing an overstressed bar restores it fully to its normal condition of perfect elasticity in both tension and compression.*

Both of these diagrams are very instructive and will bear close study. Many more such could be plotted from the tabulated results found in the

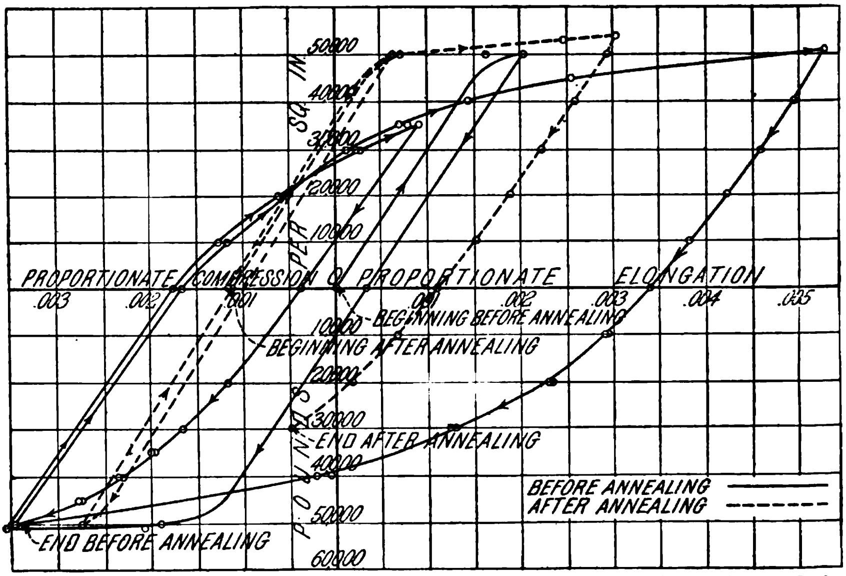


FIG. 437.—Alternate Tensile and Compressive Distortions of a Steel Bar before and after Annealing. (*Wat. Ars. Rep.* 1889.)

Reports of the Watertown Arsenal, from which the data for these were obtained.

The effect on the ultimate strength of 60,000-lb. steel of pulling it nearly to final rupture is shown in Fig. 437*a*. This also gives some idea of the homogeneity of the steel. Here, after the specimen had necked down under the tensile load, but before it broke, it was removed, a continuous screw-thread cut on it, and a series of grooves were cut in it as shown. The bar was then broken in tension at all these grooves in succession, with the results as shown in Fig. 437*a*. The original tensile strength was about 57,000 lbs. per square inch, the final breaking stress on the grooved sections was about 100,000 lbs. per square inch, while the final stress on the groove placed at the centre of the necked-down portion was 155,000 lbs. per square inch. A portion of this increase in strength is due to the normal difference between the strength of a grooved and of a parallel-sided specimen, as shown in Fig. 427. After allowing for this difference there still remains a great increase of strength due to the previous drawing out and the intervening rest the specimen had experienced.

The results of similar tests on unstressed or normal bars are shown in the original plate from which Fig. 437*a* was taken, which go to show that grooved sections on the same steel bar may develop breaking tensile stresses which differ from each other by as much as 20 to 25 per cent. No such differences

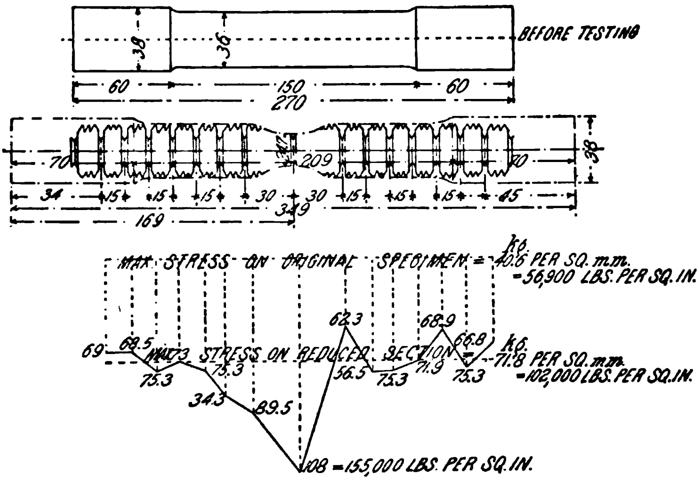


FIG. 437*a*.

would be observed on specimens with parallel sides cut from the same bar, and hence we must conclude that *results of tests on grooved sections of steel are very erratic and unreliable.*

374. Shearing Resistance of Steel.—Prof. A. B. W. Kennedy, by means of the apparatus shown in Fig. 311, obtained values of tensile and shearing resistances of various steels given in Table XXXIV.

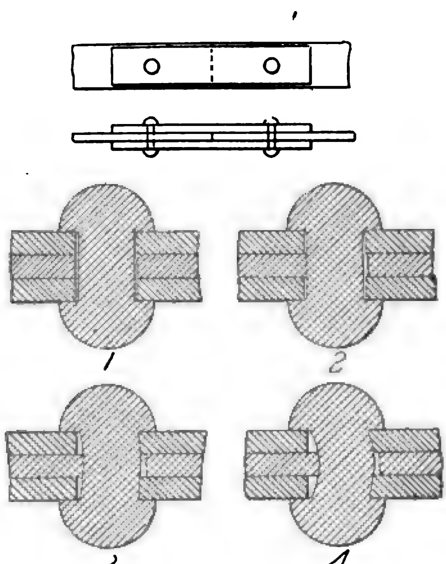
TABLE XXXIV.—SHEARING RESISTANCE OF STEEL.*

Kind of Steel.	Number of Tests Averaged.	Tension Test. Elastic Limit, Pounds per Square Inch.	Tension Test. Ult. Strength, Pounds per Square Inch.	Shearing Strength, Pounds per Square Inch.	Ratio of Shearing to Tensile Strength.
Landore Siemens steel..	2	37,500	57,000	47,500	0 829
" " " ...	2	40,000	63,500	51,000	0 800
" " " ...	3	37,000	64,000	52,000	0.811
" " " ...	6	40,600	69,000	56,000	0.807
Weardale Bessemer " ...	6	44,000	71,000	51,000	0.715
Bessemer steel..........	4	51,500	78,000	64,000	0.823
" "	4	62,000	82,000	59,000	0.721
Crucible "	2	69,500	116,000	74,000	0.632
Bessemer "	2	70,000	118,000	79,000	0.670

* From *Proc. Inst. Mech. Engrs.* 1885, p. 262.

From the above table, which is simply corroborative of a vast amount of similar data, we may reasonably use 0.8 as the ratio of shearing to tensile strength of mild or structural steel.

375. The Frictional Resistance of Riveted Joints.*—The contraction of rivets in cooling is always much more than their elastic stretch. Thus if the modulus of elasticity be taken at 30,000,000, and the elastic limit of rivet-steel at 30,000 lbs. per square inch, then the elastic stretch is 0.001 of the length. But as the contraction per degree F. is 0.0000065, it follows that $\left(\frac{0.001}{0.0000065}\right) = 154°$ F. change of temperature would bring rivets to their elastic limit if they were not allowea to contract. Evidently, therefore, all well-driven rivets in plates which are tightly clamped together when the rivet



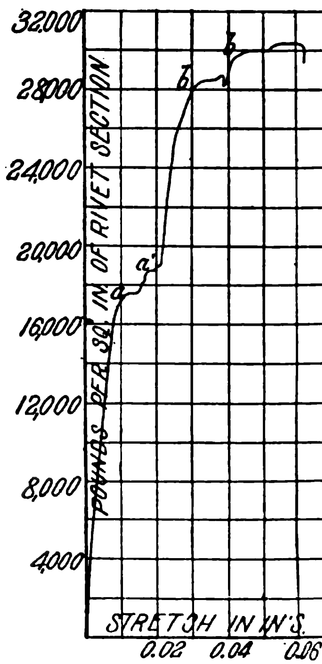


Fig. 438. Fig. 439.

Fig. 438.—Showing the Successive Stages of the Slipping of Riveted Plates. (M. Dupuy in *An. d. Ponts et Chaussées*, 1895.)

Fig. 439.—Autographic Stress-diagram of a Double-strap Butt-riveted Joint with one rivet on each side of joint, showing slips at *aa'* and *bb'*. (*An. d. Ponts et Chaussées*, vol. IX, 1895.)

is driven, and so held till the rivet cools, are left in a state of tension exceeding their elastic limits. If the coefficient of starting friction be taken at 0.4, and the elastic limit of steel rivets be taken at 30,000 lbs. per square inch, and of iron rivets at 25,000 lbs. per square inch, it follows that the frictional resistance would be 12,000 lbs. per square inch of rivet section for

* Riveted joints are a kind of structure the strength and design of which do not fall within the scope of this work. The *elements* of the strength of such structures, however, are properly treated here.

steel rivets, and 10,000 lbs. per square inch of rivet section for iron rivets, in lap-joints, and twice these amounts for butt-joints with two cover-plates, since in that case there are two frictional surfaces on each of which the full tensile stress in the rivets acts. Theoretically, therefore, we might expect a frictional resistance of 12,000 or 24,000 lbs. per square inch of rivet-surface for lap and butt joints respectively when the rivets are of steel, and of 10,000 or 20,000 lbs. when the rivets are of iron.

Since the plates are clamped together much more firmly when steam or hydraulic riveting-machines are used than when the rivets are driven by hand, so the experiments show a much higher frictional resistance for machine-driven rivets. To secure the greatest frictional efficiency the machine pressure should remain on until the rivet has cooled, but in ordinary commercial work this is seldom done.

M. Dupuy has carefully and fully investigated this question.* He shows that after cooling the rivet does not fully fill the hole, as shown in Fig. 438 (1). The first slip, therefore, when a butt-joint has one rivet on each side as shown in Fig. 438 (2), is that which brings the centre plate against the rivet. This is shown at *a* and *a′* in Fig. 439, this being a reduced autographic stress-diagram for the joint shown in Fig. 438. The slip at *a* occurred under a load of 17,500 lbs. per square inch of rivet area when the centre plate came up against the rivet on one side of the joint, and the slip at *a′* marks a similar movement on the other side at 18,500 lbs. per square inch of rivet area. After these movements had occurred the load was increased to 28,000 lbs. per square inch of rivet area, when the rivet-heads slipped on the cover-plates, as shown in Fig. 438 (3), and on the stress-diagram in Fig. 439 at *b*. Soon after the same action occurred at the other rivet, marking the deformation at *b′* in Fig. 439. All these four slips were sudden, and were accompanied by a sharp report like a pistol-shot. After the spaces had all closed up in this way the deformation was gradual, and the rivet would then be acting as a bolt, and be subjected to a shearing stress and deformation as shown in Fig. 438 (4).

376. The Stresses per Square Inch of Rivet Section at which the First Slipping Occurs, as determined by M. Dupuy, with an extensometer (called by him an elasticimeter), are given in Fig. 440. They are presented in this form in order that the relative frictional efficiencies of different methods of riveting may be read at a glance. These results were obtained by cutting the plate along the centre line of the rivets and then pulling out the two halves of these rivets as indicated in the figure. In this way the frictional resistance of the rivet-heads was correctly obtained without any complication with bearing or shearing resistance, as must always be the case when pulling actual riveted joints.

From tests on riveted joints made at the Watertown Arsenal (1882) we find the following values of frictional resistance on plates with elongated holes, hand-riveting:

* In *An. d. Ponts et Chaussées*, 7th series, vol. IX, 1895

	Frictional Resistance on one Surface in Pounds per Square Inch of Rivet Area.
Steel plates, $\frac{5}{8}$-in. iron rivets, $1\frac{5}{8}$-in. grip, lap-joint, 4 tests..............	14,550
Iron " $\frac{5}{8}$-in. " " 1-in. " " " 4 "	14,100
" " $\frac{3}{4}$-in, " " 1-in. " butt-joint, 2 cover-plates, 2 tests	9,000

It thus appears that the frictional resistance is not twice as much on a butt-joint having two cover-plates as it is on a lap-joint. The reason may

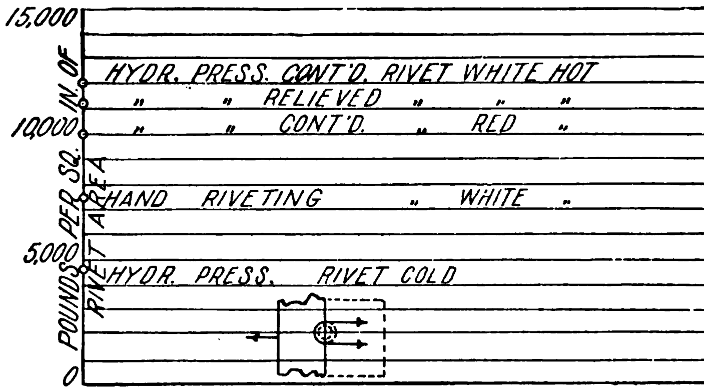


Fig. 440.—The Slipping Resistance of Steel Rivets in Pounds per Square Inch of Rivet Cross-section for Various Conditions of Driving. Each result is the overage of 25 tests on single rivets from $\frac{1}{2}$ inch to 1.2 inches diameter. Plates and rivets cut on the diametral lines and each half of rivet pulled out as shown. (M. Dupuy in *An. d. Ponts et Chaussées*, 1895, p. 105.)

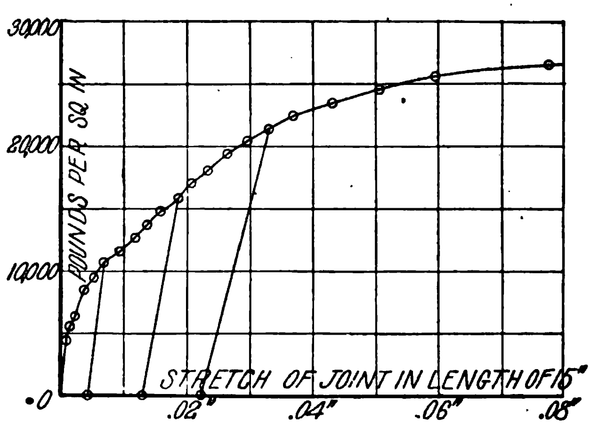


Fig. 441.—Stress-diagram of Test of Double-butt-strap Riveted Joint. Steel plates 0.662 in. thick and 20 in. wide. Thirteen $\frac{3}{4}$-in. steel rivets, machine-driven, on each side of joint. Drilled holes $\frac{13}{16}$ in. diameter. (*Wat. Ars. Rep.* 1887, p. 892.)

be that the distortion of the lap-joint increases the frictional resistance by putting an additional tensile stress on the rivet from the bending of the plates.

Since the frictional resistance is thus seen to depend directly upon the total shearing area of the rivets, whether these be in single or in double shear (although in double shear the frictional resistance on each bearing surface seems to be less than in single shear), there would seem to be no advantage in designing riveted joints for frictional resistance. It seems

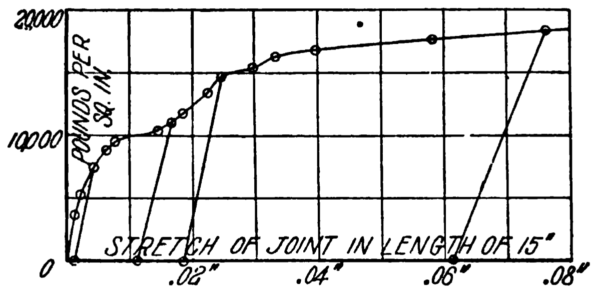


FIG. 442.—Stress-diagram of a Test of a Double-butt-strap Riveted Joint. Steel plates $\frac{3}{4}$ in. thick and 16.5 in. wide. Eleven 1-in. steel rivets, machine-driven, in three rows on each side of joint. Straps $\frac{5}{8}$ in. thick. Drilled holes $1\frac{1}{16}$ in. diameter. (*Wat. Ars. Rep.* 1887, p. 901.)

probable, however, that all riveted joints in practice do their work through friction alone, and that in no case are the rivets subjected to either shearing or bearing stress. But when the joint is dimensioned for shear, it is likely to be also properly designed for frictional resistance. In the case of double shear, riveted joints are usually proportioned for bearing stress, and here it

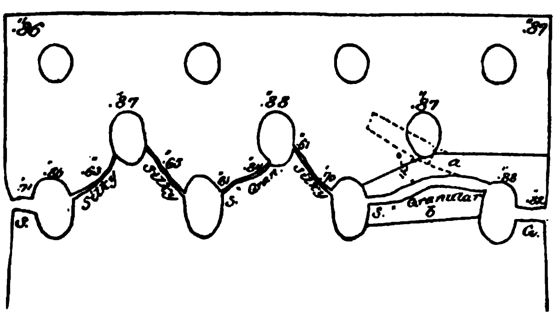


FIG. 443.—Showing Manner of Failure of a Triple-riveted Steel Plate. Figures indicate thickness of plate at that place. (*Wat. Ars. Rep.* 1887.)

would seem to be proper to make due allowance for the frictional resistance which, for all working loads, will at least greatly reduce the bearing stress.

The frictional resistance of joints containing double or triple rows of rivets cannot be observed, because all the rivets do not draw with the same tensile force, and hence the slipping is progressive and without any sudden manifestation. This is well shown by Figs. 441 and 442, which are characteristic of a great many such tests made at the Watertown Arsenal. Evi-

dently it would be impossible here to locate the point of initial slipping. This explains Prof. Kennedy's discrepant results on this class of joints, as recorded in the Proceedings of the Institution of Mechanical Engineers (London) for the years 1885 and 1888. He here records the loads for which "visible slip began"; but as he used only a hand magnifying-glass, and the movement was a gradually progressive one, it would be quite impossible to obtain consistent or rational results. In fact, for such joints no such stage of the test exists, since the slipping does not occur over the entire joint at any one time.

377. The Bearing Resistance of Steel and Iron Plates is shown in Fig. 444. This is seen to increase directly with the distance of the hole from

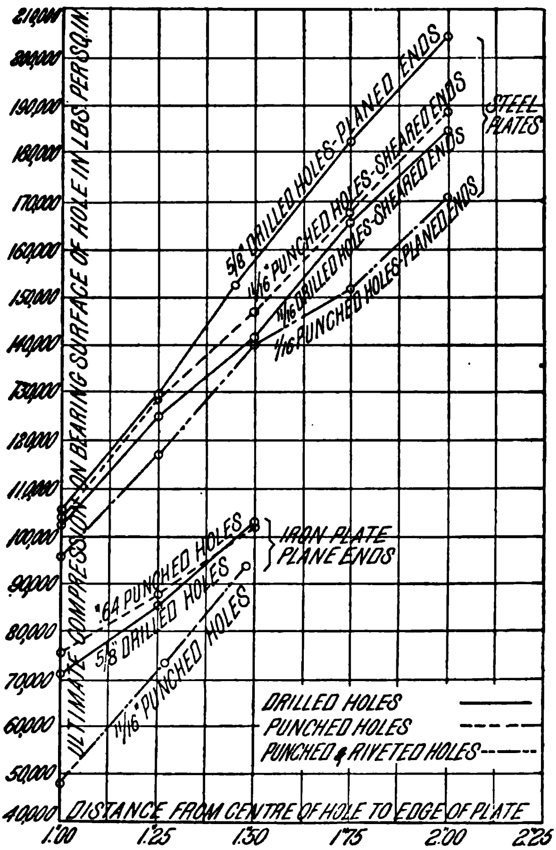


FIG. 444.—Bearing Resistance on Rivet-holes at Rupture by Tearing Out of Hole. The steel plates were of 60,000 lbs. tensile strength. (*Wat. Ars. Rep.* 1882.)

the edge of the plate. When this distance agrees with ordinary practice the resistance is so high that it would seem a working bearing stress of

16,000 lbs. per square inch might be employed for iron, and of 24,000 lbs. per square inch for steel, plates. The stresses here plotted were the bearing stresses at rupture, where the plates had so reduced in thickness as to

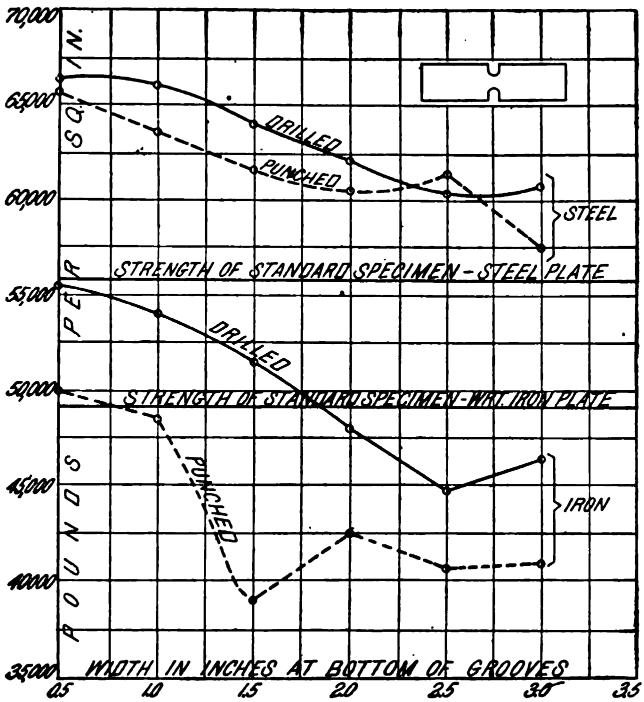


FIG. 445.—Variation in Strength of ½-in. Plate for Varying Widths at Bottom of Groove. Each plotted result is the mean of from three to eight tests. (*Wat. Ars. Rep.* 1882.)

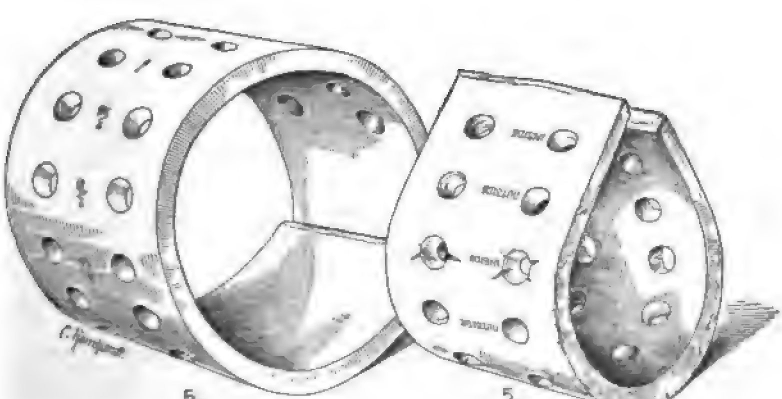


FIG. 446.—Effects of Punching, Reaming, and Shearing. No. 5 has punched holes and sheared edges. No. 6 has punched and reamed holes and planed edges. (*Engr. News*, vol. XXXIII. p 291.)

destroy all frictional resistance. Much more, then, could high *working* stresses be employed, since for these the frictional resistance is very great. The author believes that the ordinary rules for proportioning riveted joints might well be modified so as to allow higher bearing stresses, especially on steel. With wrought iron, especially when the stress is transverse to the

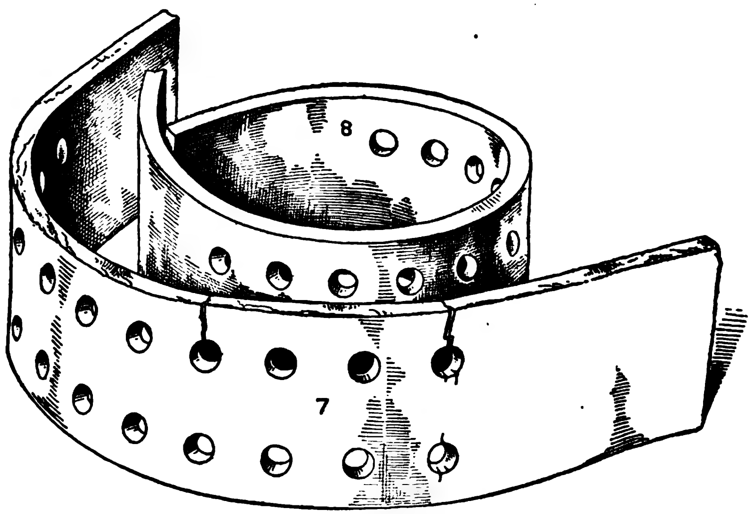


FIG. 447.—Effects of Shearing and Punching on Bessemer-steel Plate $\frac{1}{2}$ in. thick. Specimen 7 had sheared edges and punched holes. Specimen 8 had planed edges and drilled holes. (*Engr. News*, vol. XXXIII. p. 291.)

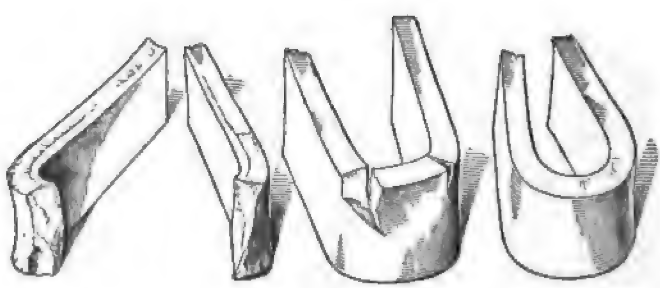


FIG. 448.—Showing that Injury in Case of Shearing and Punching comes from the Compression of the Metal Necessary to Produce the Shear. Nos. 1 and 2 were bent cold, with the compression edge on convex side; No. 3 was bent with compression edge on concave side. (*Engr. News*, vol. XXXIII. p. 290.)

fibre, more care must be exercised, as this material is liable to be very weak in this direction.

378. The Tensile Strength of Grooved Plates is a measure of the tensile strength of a riveted joint when failure occurs by tearing the plate. This strength is found to be a function of the width of the net section at the

bottom of the groove, as well as of the method of obtaining the hole, and of the character of the material. These effects are all shown in Fig. 445 for ¾-in. plates of wrought iron and of 56,000-lb. steel. The steel, being more ductile, is stronger in the grooved than in the plain (standard) section, while the reverse is the case with wrought iron, except with drilled specimens, where the width of the net section was less than 1¾ in.

379. The Injurious Effect of Punching and Shearing is Found on the Compressed Side Only.—In punching and shearing cold metal it seems to be the compression produced by the shears or by the die-plate which injures the metal here and makes it brittle by cold flowing. This is clearly shown in Figs. 446 and 448. Thus in Fig. 446 (5), when the punched plate is bent with the punch (or upper) side in tension, no cracks appear about the punched holes, but when the die (or lower) side of the hole is on the tension side of the bent plate, Fig. 446 (5), many cracks appear and radiate from such openings. When these holes are reamed, however, as in Fig. 446 (6), no such cracks develop.

Similarly, in Fig. 448, when a bar is cut off with two sheared edges, and if both pressed corners (from having turned the plate over) are on the same side of the bar, and this side be put in tension, as in Fig. 448 (1), then it breaks as shown. If these pressed edges are on opposite sides of the bar, it breaks only at that edge, as in (2), while if both sheared edges have been planed, as in (3), it bends without cracking.

380. The Avoidance of Scarfed Joints.—This may be effected as shown in Fig. 449. Here lap-joints are used in one direction and butt-joints in

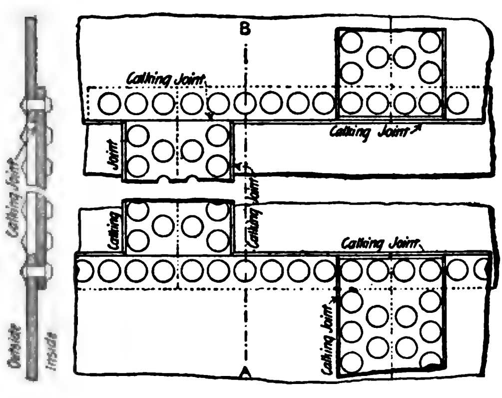


FIG. 449.—Proper Method of Joining Riveted Work in Stand-pipes and Boilers when single butt-straps are used. (*Engr. News*, vol. XXXIII. p. 290.)

the other, with one cover-plate. This requires twice as many rivets in this direction, but it makes a much neater and stronger construction and it avoids the heating of one corner of every plate for the purpose of scarfing it down to a thin edge, as must be done where three plates come together in a

lap-joint. In the figure all the outer edges are planed to a bevel for calking.*

381. Steel Specifications.—For three sets of specifications, by a Committee of the American Society of Civil Engineers, by the Association of American Steel Manufacturers, and by Mr. H. H. Campbell, Supt. Steel Works at Steelton, Pa., see Appendix D.

382. The Influence of the Form of the Thread on the Strength of Screw-bolts.—This subject has been investigated by Prof. Martens,† and his results are here given.

Two grades of mild steel were used for these bolts, all of which were cut from round bars originally 35 mm. (1.4 in.) in diameter. The softer material, having a tensile strength of 53,500 lbs. per square inch, was used for screw-bolts approximately one inch in diameter, and the harder material, having a tensile strength of 62,000 lbs. per square inch, was used for the screw-bolts, which were reduced to approximately one-half inch in diameter. Four such bolts were made of each of these sizes for each of the four styles of thread shown in Fig. 450, making in all 32 bolts with screw-threads

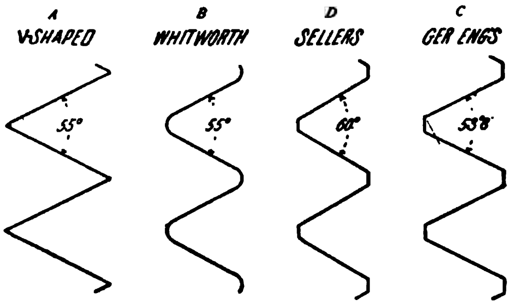


Fig. 450.

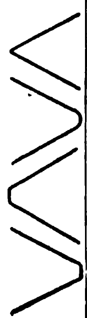
which were tested. Two of each of these sets were tested in plain tension, the pulling force being applied to the inner face of the nut at one end, and increased until rupture occurred. The other two bolts of each set were tested also in tension, but under a torsional action resulting from the continuous turning of the nut as the load increased to rupture. In this case the distortion resulting from the permanent elongation of the bolt was nearly all taken up by the movements of the testing-machine, the distortion taken up by the turning of the nut being the least possible to maintain a continuous torsional action at this point.

The same bars were also tested as plain tension-test specimens with cylindrical bodies, and again with grooves turned into them of the same shape as

* See *The Locomotive* for Nov. 1896 for a full discussion of quadruple-riveted, double-butt-strap joints having an efficiency of 95 per cent.

† At the request of the German Society of Civil Engineers. The results were published in *Zeits. d. Ver. Deutsch. Ing.* for April 27, 1896. The abstract here given was made by the author and published in the *Digest of Physical Tests* for July 1896.

TABLE XXXV.—ABSOLUTE AND RELATIVE STRENGTH OF THREADED BOLTS IN POUNDS PER SQUARE INCH. (MARTENS.)

Kind of Thread.	Form of Base of Thread	Diameter = 1 inch.								Diameter = ½ inch.							
		Stress Applied by			Proportion.					Stress Applied by			Proportion.				
		Machine.		Nut.	Test Bar = 100.			$f_g = 100$.		Machine.		Nut.	Test Bar = 100.			$f_g = 100$.	
		G.* f_g	T.* f_t	T. f_{tn}	f_g	f_t	f_{tn}	$\frac{f_t}{f_g}$	$\frac{f_{tn}}{f_g}$	G. f_g	T. f_t	T. f_{tn}	f_g	f_t	f_{tn}	$\frac{f_t}{f_g}$	$\frac{f_t}{f_{tn}}$
(a) Sharp under 55°		62,430	61,580	49,920	116.8	115.2	93.4	98.9	80.0	71,100	70,400	62,720	114.9	114.0	101.4	99.2	88.2
(b) Whitworth....		62,160	61,300	44,800	116.2	114.6	83.8	98.9	72.2	67,000	69,400	58,880	109.4	112.2	95.2	100.8	86.9
(d) Sellers........		60,800	60,020	52,330	112.8	112.2	97.9	99.6	86.9	70,250	68,120	62,720	113.6	110.1	94.5	96.9	83.2
(c) German Soc. of Eng'rs.....		60,730	61,160	47,640	113.6	114.4	89.1	100.7	78.6	69,260	73,670	62,720	112.0	119.1	101.4	106.4	90.8
(e) Normal Test Bar........			58,480				100				61,860				100		

* G. = grooved ; T. = threaded,

the corresponding screw-threads, leaving the same diameter at the bottom of the groove as obtained at the base of the threads. The actual and comparative average results of all of these tests are given in the following table, from which the following conclusions may be drawn:

1. When subjected to plain tension both the screw-threads and the grooved sections were stronger than the plain bars of the same net area of cross-section, this excess of strength having an average value of about 14 per cent. This excess of strength is due to the re-enforcing action of the shoulder in the case of the groove, and of the threads themselves in the case of the screw.

2. There is no very marked difference in the average strength of the bolts on which the several styles of thread were cut, the perfectly sharp groove shown at *A* being slightly stronger than the others.

3. The weakening effect of the turning of the nut under stress at rupture is much less than might have been predicted, when the distortion of the screw below the nut by permanent elongation is taken into consideration. The tests indicate for this case a strength of the one-inch bolts about 20 per cent less than that of the plain bars, and of the one-half-inch bolts about 15 per cent less than that of the plain bars.

4. In general it may be said that the turning of the nut upon the bolt at rupture reduces the strength of the net section of the bolt by about 30 per cent.

5. It is very probable that the four forms of screw-threads here shown would show very different results under fatigue tests from repeated stresses, and also for static loads on high-carbon steel. Under repeated loads and under shock it is probable that the sharp re-entrant angle shown in Fig. 450 *A* would develop incipient cracks much earlier than either of the other forms, and that probably the Whitworth thread, shown in *B*, would be the last to develop this kind of weakness, either with soft metal under repeated loads or with high-carbon steel under static loads. No such tests have as yet been made. It is to be hoped that this subject will soon be investigated, as it is of far more importance than the mere matter of static strength.

CHAPTER XXVII.

THE FATIGUE OF METALS.

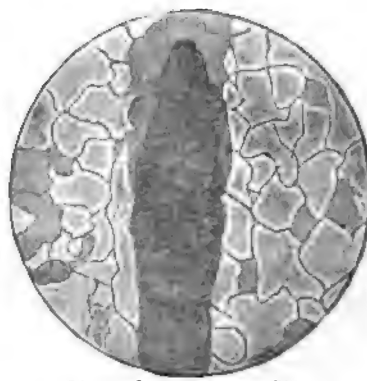
383. Fatigue Defined.—It has been found from experiment that metals will fail under loads much less than their ultimate strength when such loads are repeated or reversed many thousands or perhaps millions of times. It has been commonly supposed that these repetitions or reversals caused a general deterioration of the metal so stressed, so far as its cohesion is concerned, which deterioration has been known by the term *fatigue*. It is now known, however, that no such general deterioration takes place, but that some of the millions of incipient defects or "micro-flaws" in the specimen gradually extend their weakening influence, in an irregular plane of cross-section, which ultimately becomes the plane of rupture, while the metal immediately adjacent to this plane remains perhaps wholly uninjured. In fact no tests of metal, on specimens as closely adjacent to such planes of rupture as it is possible to procure them, have ever shown any deteriorating effects of the repetitions or reversals of stress to which this metal had been subjected. The word "fatigue," therefore, is scarcely the proper term to apply to this class of failures. *The gradual fracture of metals* would be a more truly descriptive term to use.

384. The Micro-flaws in Steel have been studied exhaustively by Mr. Thos. Andrews, F.R.S., M. Inst. C.E. of Sheffield, England, and described in *Engineering* of July 10, 17, and 24, 1896. Some of his illustrations are here reproduced in Fig. 451. The large flaws in Nos. 3, 4, 5, and 6 are due to small blowholes, while the dark intercellular spaces in Nos. 1 and 2 are largely composed of the sulphide of iron, which, so far as it destroys the continuity of the crystals, makes the iron weak and brittle. These and similar incipient faults, of which there are probably scores in every square inch of any iron or steel cross-section, are doubtless the initial cause of the weakness developed by repeated loadings. These breaks in the continuity of the metal cause the stress to be concentrated at their edges, and the constant variation of this stress, near or at the elastic limit, with its accompanying molecular movements, gradually extends the fracture. Evidently there can be no regularity of action of such causes, and hence no very rigid rule or law for such failures. Even two specimens cut from the same bar may act very differently, to say nothing of specimens made by the same processes at

1. Micro-flaws × 250; Sulphur = 0.25 per cent.

2. Micro-flaws × 200; Sulphur = 2.00 per cent.

3. Micro-flaws × 250. Siemens-Steel Boiler-plate.



4. Micro-flaws × 250. Siemens-steel Propeller-shaft

5. Micro-flaws × 400. Bessemer-steel Railway-axle.

6. Micro-flaws × 250. Bessemer-steel Rail.

FIG. 451.—Views of Internal Micro-flaws in Steel. (Andrews in *Engineering*, July 10, 1896.)

different times and at different works, or of specimens made by different processes and having different chemical compositions. Evidently the results of fatigue tests would be extremely various, and this is the experience of all the experimenters in this field of investigation.

385. Wöhler's Tests.—The first systematic study of the fatigue of metals was made by Wöhler from 1849 to 1870 for the German government, and

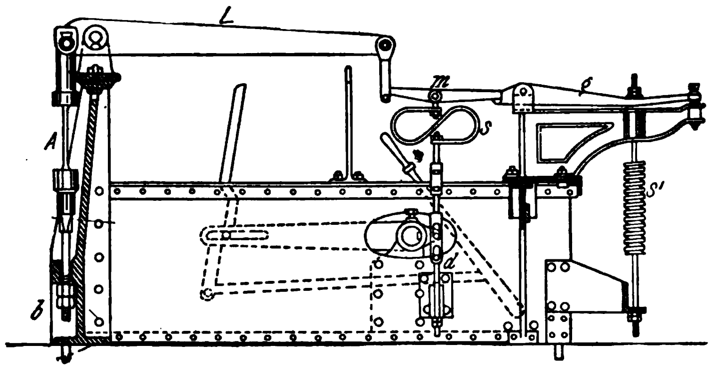


FIG. 452.—Wöhler's Machine for Repetitions of Tensile Stress.

these were continued after his death by Spangenberg. As Wöhler's tests have become historically famous, his appliances are here described.

For Repeated Tensile Stresses Wöhler used the apparatus shown in Fig. 452. Here the specimen A is stressed through the lever L and spring s

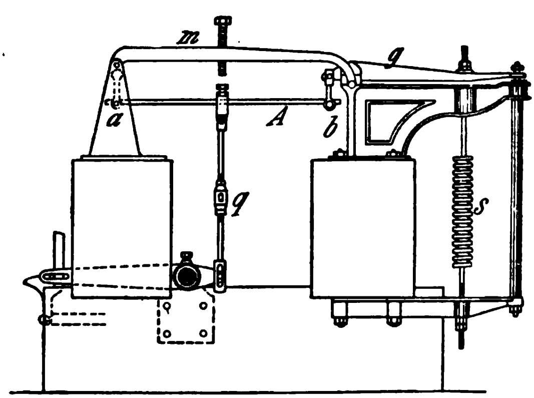


FIG. 453.—Wöhler's Machine for Repetition of Bending Stress.

acting on the auxiliary lever m. The pull of the spring s is measured by the starting of the adjusted calibrated spring s' through the terminal lever g. The nut at the rod d is adjusted to give the minimum load on the spring s by starting the spring s' when adjusted to a particular tension, and the cam-movement of d to its extreme downward position is made to give the requisite maximum stress in the specimen by adjusting the spring s' so as just

to lift at this position of d. The rod is adjustable by means of a turnbuckle. In this way the bar A can be stressed in tension between any chosen limits.

For Repeated Bending Stresses Wöhler employed the machine shown in Fig. 453. Here the specimen A is bent downwards by the adjustable rod q attached to the rocking lever below. If the load is not to be wholly removed each time, a residual deflection is maintained by means of the abutting screw in the lever m. Both the maximum and the minimum loads are fixed by means of the calibrated spring s acting on the attached lever g.

For Reversals of Bending Stress Wöhler made use of the apparatus shown in Fig. 454. Here two test-bars AA are attached by a driving fit to

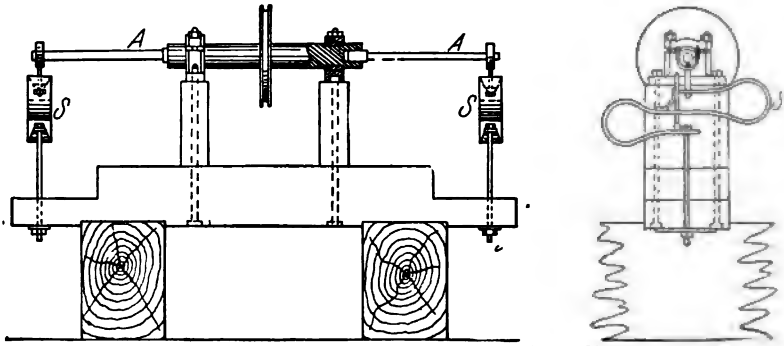


FIG. 454.—Wöhler's Machine for Reversals of Bending Stress.

the central axle, which is rotated by a belt and pulley. The ends of the test-bars are held down by the calibrated springs ss, so that the bending stresses are reversed at every revolution. Of course the test-specimens are trued up to run truly after driving and before loading.

For Repetitions of Torsional Stress Wöhler devised the machine shown in Fig. 455. Here the specimen A is fastened to the moving lever L at one

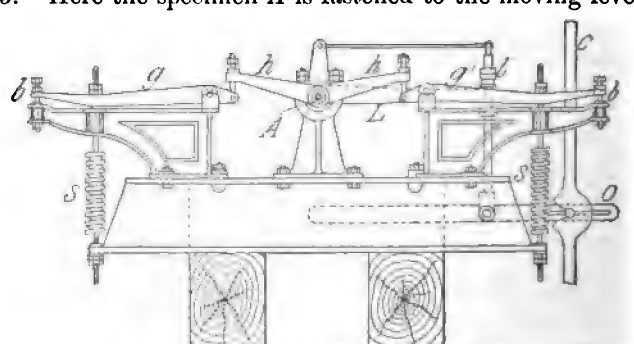


FIG. 455.—Wöhler's Machine for Repetition of Torsional Stress.

end and to the resisting levers hh at the other. The lever L is actuated by the connecting-rod l and the lever O, which in turn is moved by the recip-

rocating-bar C. If the bar is stressed in opposite directions, then both the levers g and g' are in use, and the calibrated springs ss act to limit the torsional moment to the required amount as before.

386. The Results of Fatigue Tests.—The most careful and complete set of fatigue tests under repeated stresses was made by Bauschinger. His results on mild-steel plates are shown in Fig. 456. For this material, which had an ultimate strength of 64,000 lbs. per square inch the repetition limit was found to be about 35,000 lbs. per square inch, or about the elastic limit

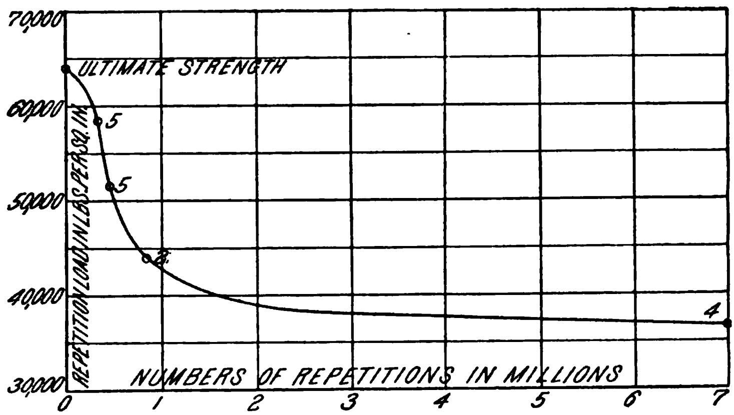


FIG. 456.—Bauschinger's Fatigue Tests on Mild-steel Plates under Tensile Stress Repeated from Zero. Attached figures indicate number of tests averaged.

of the material. This material was very uniform in quality and gave quite consistent results. In general the results of such tests are very discrepant, as should be anticipated from the nature of the causes operating to produce the final fracture.

In Fig. 457 are given the results of a series of tests by reversed bending stress on various grades of steel and on cold-rolled wrought-iron bars. As the steel bars seemed to give way under about the same stresses, irrespective of their several elastic limits and ultimate strengths, they have here all been averaged to bring them into comparison with the tests on the wrought-iron bars. These results seem to be favorable to wrought iron rather than to this particular kind of steel. As both the phosphorus and sulphur were pretty high in all these steel bars (see Figs. 76 and 451), the weakening effects of these may account for the relatively poor showing of steel in this series of tests. There is no doubt, however, that the best grades of wrought iron have this advantage over steel, that an incipient fault or fracture does not so readily extend itself across the section, but is more likely to be stopped by the slag impurities which separate the filaments. In the more homogeneous and more perfectly crystallized steel a micro-flaw more readily extends throughout the section.

387. Limits of Maximum and Minimum Stresses for an Indefinite Number of Repetitions.—Wöhler's tests revealed the fact that for an indefinite

number of repetitions of the maximum load this maximum itself could be increased if a portion of the stress were left on. Thus his tests on spring-steel, which had a static tensile strength of 124,000 lbs. per square inch, gave results as plotted in Fig. 458. When the load was wholly removed each time, the maximum load which could be repeated many millions of times

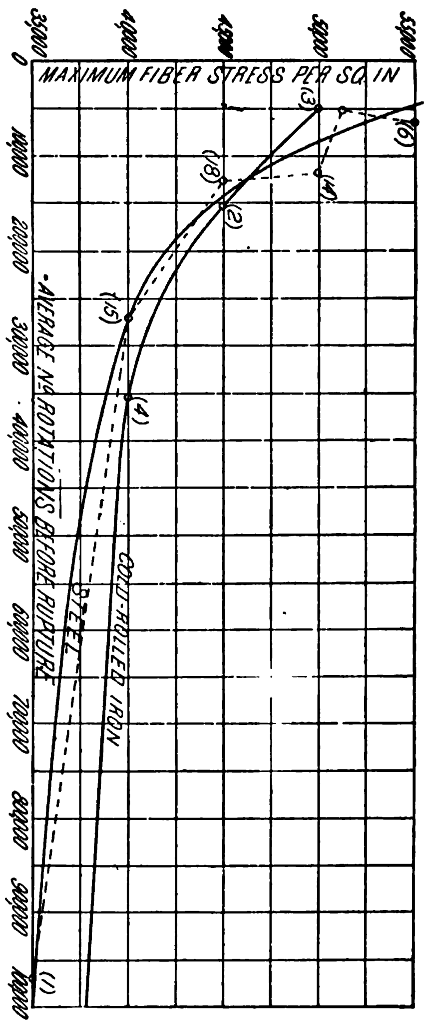


FIG. 457.—Fatigue Tests on Cold-rolled Wrought-iron and on Gautier Steel (from 0.16% to 1.09% Carbon) under Reversed Bending Stresses. (*Wat. Ars. Tests*, 1892 and 1894.) For tensile stress-diagrams of the Steel see Figs. 401, 402, and 403.

was 67,000 lbs. per square inch, which is marked p_1 in the figure. When 24,000 lbs. stress per square inch remained on each time, the maximum load could be raised to 75,000 lbs. per square inch, and repeated an unlimited number of times. When there was 35,000 lbs. stress left on, the maximum load could be raised to 86,000 lbs. per square inch; when the

minimum was 56,000 lbs. the maximum was 96,500 lbs., and when the minimum was 70,000 lbs. per square inch the maximum could be raised to 108,000 lbs. per square inch, with an indefinite number of repetitions.* In

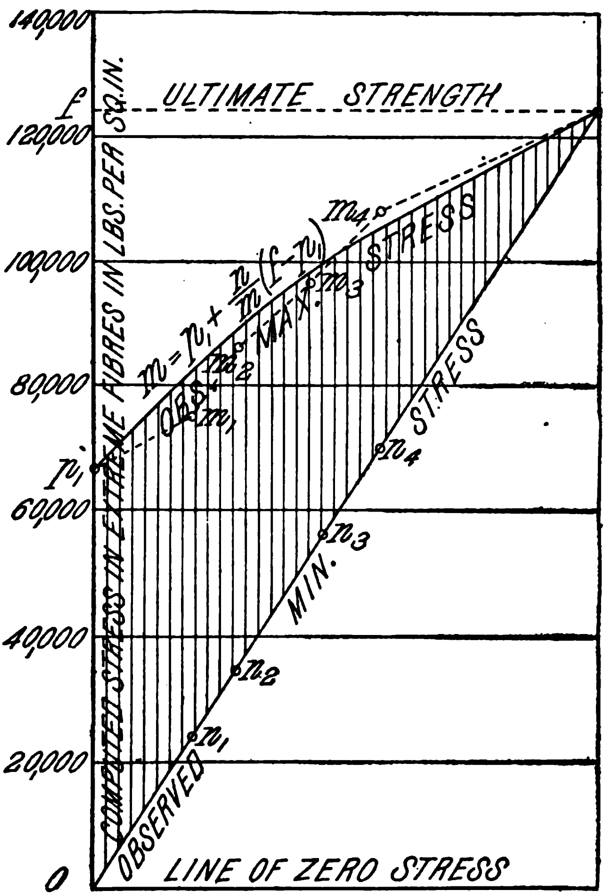


FIG. 458.—Results of Wöhler's Fatigue Transverse Tests on Spring-steel. The shaded area is the field in which the material may be worked indefinitely.

Fig. 458 these minimum values are plotted upon a straight inclined line, and the corresponding maximum values, plotted to the same scale, fall in the broken dotted line.

These and many other similar series of tests on other grades of steel and on wrought iron led to a formula by Launhardt which may be written

$$m = p_1 + \frac{n}{m}(f - p_1), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

* See also Wöhler's results in Unwin's *Testing of Materials of Construction*, p. 368.

in which $m =$ maximum stress;
$p_1 =$ "repetition limit" when $n = 0$;
$n =$ minimum stress;
$f =$ ultimate static strength.

The locus of this curve is given as a full line in Fig. 458, and the area included between this and the minimum line is shaded, and may be considered as representing the field across any part of which this material could be stressed and relieved an indefinite number of times.

388. Limits of Maximum and Minimum Stresses when these are of Opposite Kinds.—When the stress is partly or wholly reversed an indefinite number of times, the working field is widened and the upper limit correspondingly reduced. This condition is shown in Fig. 459, the limiting

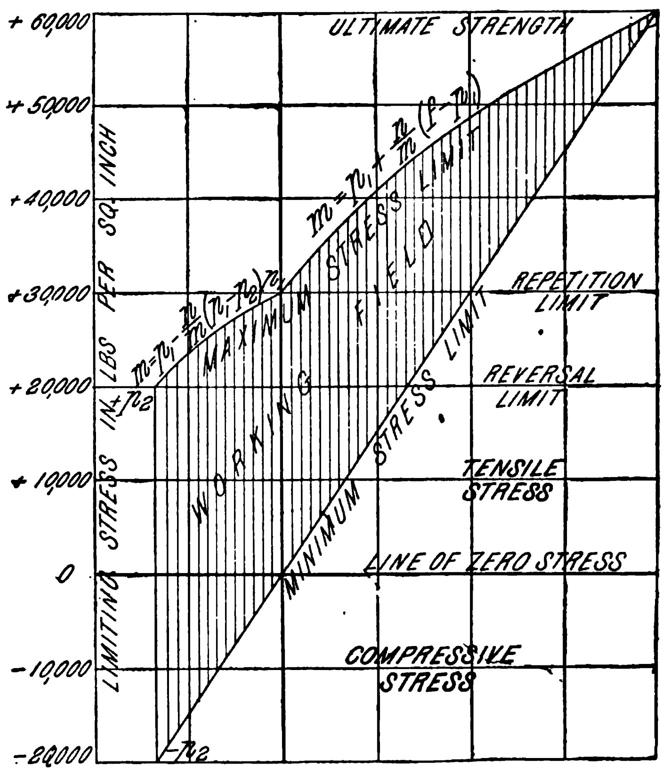


FIG. 459.—Typical Fatigue Diagram of Limiting Stresses for 60,000 lbs. Steel for an Infinite Number of Repetitions or Reversals of Stress.

case being when the stress is wholly reversed each time, when the minimum stress numerically equals the maximum stress. These limits are marked $+p_2$ and $-p_2$ in the figure, and are here called the "reversal limits." *

* These terms, *repetition limit* and *reversal limit*, were coined by the author for these values in his paper on this subject in *Jour. Assoc. Eng. Socs.*, vol. VII, 1888.

The formula for the value of the larger stress in terms of the smaller, and of these limits, p_1 and p_2, was proposed by Weyrauch, and is

$$m = p_1 - \frac{n}{m}(p_1 - p_2). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The loci of both of these equations are drawn in Fig. 459, and the working field indicated by them is shaded. Here, however, the material is supposed to be 60,000-lb. structural steel, and p_1 is taken as one half the ultimate strength, or 30,000 lbs. per square inch, and p_2 as one third the ultimate strength, or 20,000 lbs. per square inch, these being about the values of both of these limits as determined by all the fatigue tests which have ever been made.

389. A New and Universal Formula for Dimensioning.—As shown by Fig. 458, a straight line would fairly fit the observed maximum stresses for the given minimum stresses when these also are plotted on a straight line. From Fig. 459, also, it would seem unreasonable to have a sudden change of law when the minimum stress passes through zero. Furthermore, there is no theoretical basis for the particular formulæ, (1) and (2), which give these curves. It would therefore seem to be more rational, and fit the facts quite as well, to make these upper limits fall into a straight line, as shown in Fig. 460. By so doing we obtain a *single formula for both repeated and for reversed loads*, whereas now two formulæ are employed. To derive the formula for this upper limit we have, from experiment:

$$\begin{aligned}
&\textit{Static load-limit} &&= f &&= \textit{ultimate strength;}\\
&\textit{Repetition limit} &&= p_1 &&= \tfrac{1}{2}\ \textit{ultimate strength;}\\
&\textit{Reversal limit} &&= p_2 &&= \tfrac{1}{3}\ \textit{ultimate strength.}
\end{aligned}$$

Hence, when the ultimate limits are reduced to working limits, we will suppose that p_1 reduces to a, Fig. 460, and all other parts in proportion, giving

$$\left.\begin{aligned}
&\textit{Working static-load stress} = 2a;\\
&\textit{Working live-load stress} \quad = a;\\
&\textit{Working reversed stress} \quad = \tfrac{2}{3}a.
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

To find the equations of the total working stress in terms of the maximum and minimum total stresses on any member:

Let L = total live-load stress on any member;
D = " dead-load " " " "
A = area of cross-section of the member;
p = maximum stress in the member per square inch for both dead and live loads;
a = working stress for live loads.

Then we have, from Fig. 460,

$$\overline{nl} = \text{dead-load stress per square inch} = \frac{D}{A};$$

$$\overline{mn} = \text{live-load stress per square inch} = \frac{L}{A};$$

$$\overline{ml} = \text{total stress per square inch} = \frac{D+L}{A} = p.$$

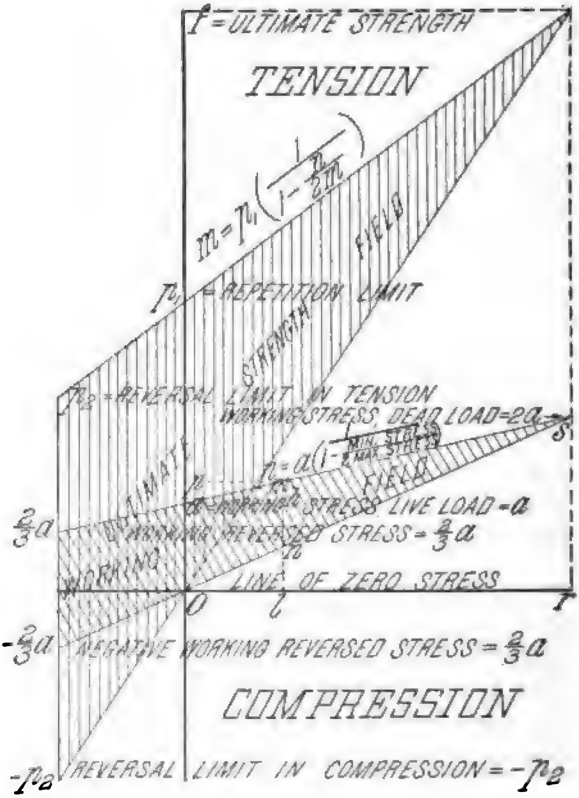


FIG. 460.

And since $\overline{rs} = 2a$, we have, from the figure,

$$p = \overline{Op} = \overline{Oa} + \overline{hm} \quad \text{and} \quad \overline{hm} = \frac{\overline{ln}}{\overline{rs}}(\overline{rs} - \overline{Oa});$$

$$\therefore p = a + \frac{\left(\frac{D}{A}\right)a}{2a} = a + \frac{D}{2A}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

But $A = \frac{D+L}{p}$; hence we have

$$p = a + \frac{Dp}{2(D+L)}, \quad \text{or} \quad p = a\left(\frac{D+L}{D+L-\frac{D}{2}}\right) = \frac{a}{1-\frac{D}{2(D+L)}};$$

and finally

$$p = \frac{a}{1-\frac{\text{min. stress}}{2\text{ max. stress}}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

This formula may be used in place of both Launhardt's and Weyrauch's equations ((1) and (2)), since it applies equally well to stresses of the same or of opposite kinds, by paying attention to the sign of the minimum stress. When the minimum stress becomes negative the sign of the second term in the denominator changes to plus, thus reducing p below a.

Another argument in favor of this formula lies in the fact that it is the same as the old rule of using twice the factor of safety for live as for dead loads, as will now be shown.

With the same notation as above, we have

$$A = \frac{L}{a} + \frac{D}{2a} = \frac{2L+D}{2a},$$

also

$$p = \frac{L+D}{A}.$$

Substituting the value of A, we have

$$p = \frac{2a(L+D)}{2L+D} = \frac{a}{1-\frac{D}{2(L+D)}} = \frac{a}{1-\frac{\text{min. stress}}{2\text{ max. stress}}}. \quad . \quad . \quad (6)$$

We find, therefore, that the past practice founded on experience, and the fatigue experiments, all agree and are all expressed in this one formula which is universal in its application to stresses of the same and of opposite signs. Its use is more laborious than those hitherto used, as given in equations (1) and (2), only in requiring a division in place of a multiplication; but as such work is now done wholly by the slide-rule, even this objection is removed.

CHAPTER XXVIII.

STRENGTH OF THE COPPER-ZINC-TIN ALLOYS.

COPPER.

390. Strength of Copper.—The first and most general error to guard against in the matter of the strength of copper and its alloys is that of ignoring the mechanical treatment to which the material has been subjected. Thus in the case of copper plate, as shown by Fig. 461, a hot-rolled plate has an elastic limit of only some 7000 or 8000 lbs. per square inch, with an elongation of 50 per cent, while the same plate, cold-hammered, has an elastic limit of over 20,000 lbs. per square inch, with an elongation of 30 per cent. Both have an ultimate strength of about 33,000 lbs. per square

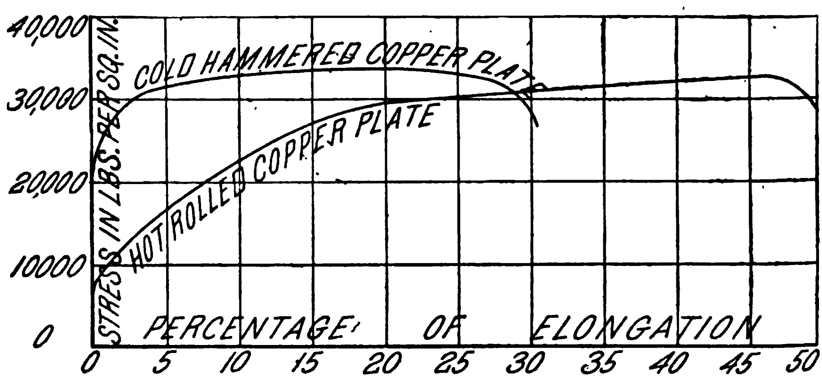


Fig. 461.—Typical Stress-diagrams of Copper Plate ½ in. thick. (Martens, *Berlin Testing Lab. Communications,* 1894.)

inch. When simply cast, without rolling or forging, both the elastic limit and the ultimate strength are much less, but copper is seldom or never used in this way.

Drawn copper wire has an elastic limit of about 25,000, with an ultimate strength of some 35,000 lbs. per square inch, as shown in Fig. 462, with an elongation of about 30 per cent.

If the strength of copper be computed on the actual section at all stages of the test, and if the strength so computed be plotted to the diminishing cross-section, the results will plot in a straight line, as shown in Fig. 463.

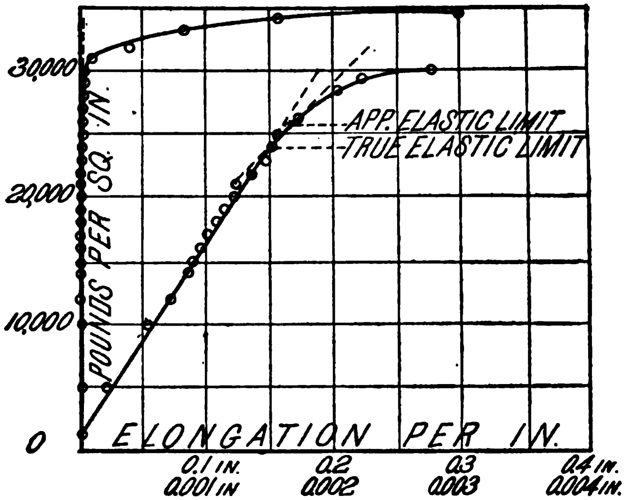


FIG. 462.—Typical Stress-diagram of Drawn Copper. (*Wat. Ars. Rep.* 1886, vol. II. p. 1673.)

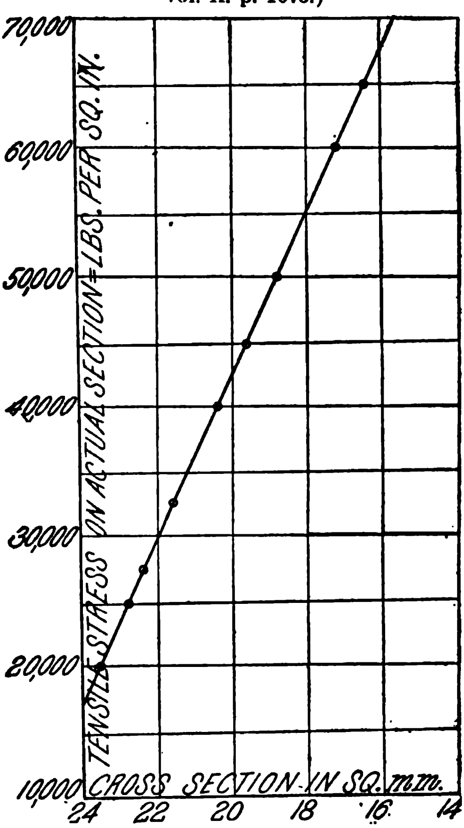


FIG. 463.—Showing a Linear Relation between Reduction of Area of Section and Stress per Square Inch of Actual Section of Rolled Copper Plate ½ inch thick. (*Rep. Fr. Com.*, vol. III, Pl. VI.)

That is to say, when copper is cold-drawn, its strength per square inch regularly increases up to rupture, when its strength per square inch of actual section is some 70,000 lbs. per square inch.

391. Annealing or Softening Hard-drawn Copper Wires or Plates.—Unlike steel, copper is softened by quenching in water from a sufficiently high temperature. The softening effect is due, however, rather to the temperature attained than to the manner of cooling. At least the sudden cooling does not prevent the softening. The annealing temperature is about 750° F., as shown in Fig. 464.

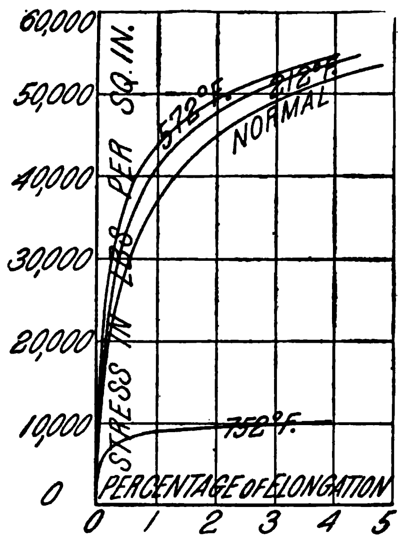


FIG. 464.—Effects of Heating to given Temperatures, and then Quenching in Water, Hard-drawn Copper Wires. (Martens, *Berlin Testing Lab.*, 1894, Pl. I.)

392. The Strength of Brass.—Brass is an alloy of copper and zinc. The mechanical properties of all possible compositions are given in Fig. 465, these applying in a general way to cast forms only. Either hot or cold forging or rolling will greatly change these properties. Thus the strength of very hard-drawn brass wire or hard-rolled brass plate may have a tensile strength of over 60,000 lbs. per square inch, with an elastic limit about the same, as shown in Fig. 466. Annealed brass plates or wires, however, have an elastic limit of only about 10,000 lbs. per square inch.

Brass is much harder than copper, as shown in Fig. 465, by the "crushing strength" diagram, this rising from 28,000 lbs. for 100 per cent copper to 120,000 lbs. per square inch for 50 per cent copper. It is this property of increased hardness which makes brass so much more useful than copper in the arts. The conductive capacity of brass is, however, much less than that of pure copper, it falling from 0.90 for pure copper to 0.20 for 70 per cent copper.

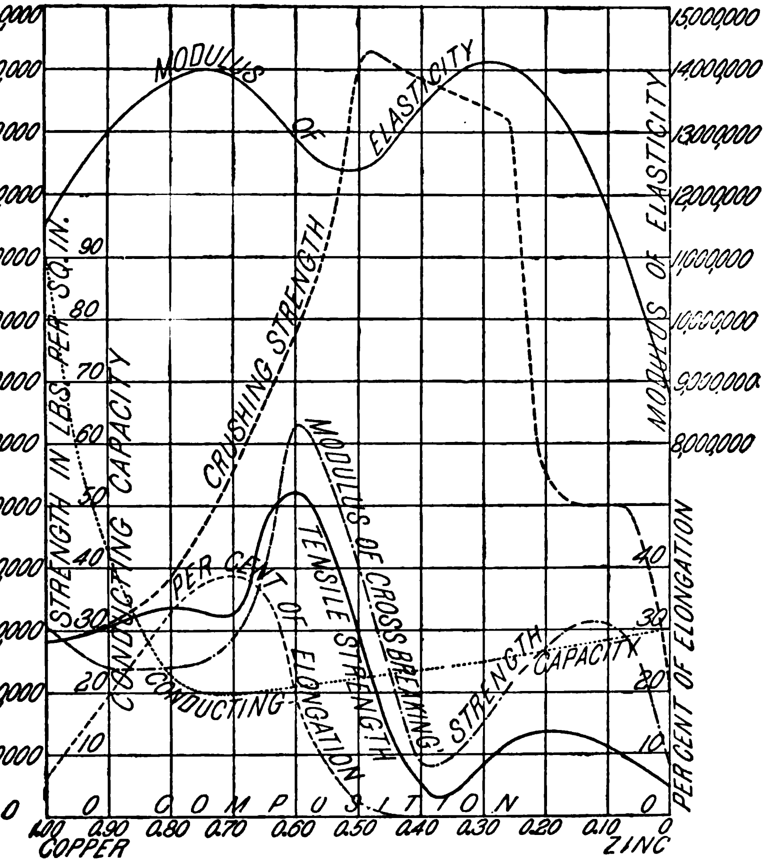


Fig. 465.—Properties of Cast Brass for Varying Proportions of Copper and Zinc. The "composition" argument gives the proportions of copper. (Data from *U. S. Test Board Rep.* 1881, vol. II.)

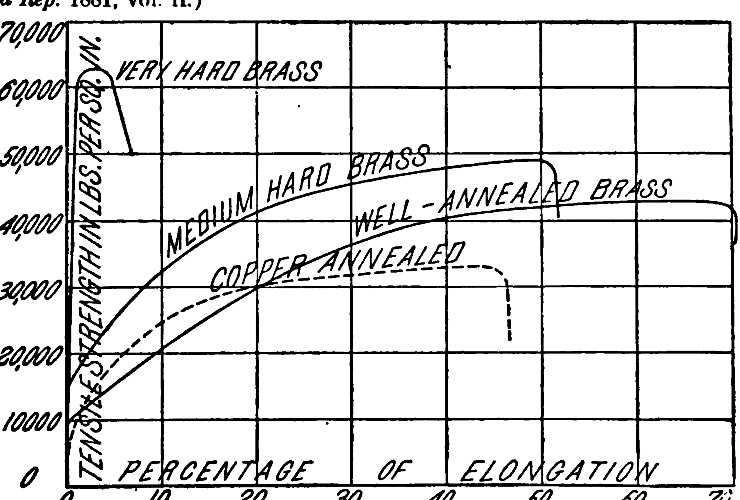


Fig. 466.—Stress-diagrams of Rolled Plate of Brass and Copper, having the Composition Cu 67. Z 33. (*Fr. Com. Rep.*, vol. III, Pl. V.)

The most generally useful brass composition is from 60 to 70 per cent copper and 40 to 30 per cent zinc, as is fully shown by Fig. 465.

By rolling to thin plates, especially by cold-rolling, the strength of brass may be greatly increased at the expense of the ductility. The simultaneous qualities of strength and ductility which may be expected from brass which, in the form of a casting, has a tensile strength of 35,000 lbs. per square inch

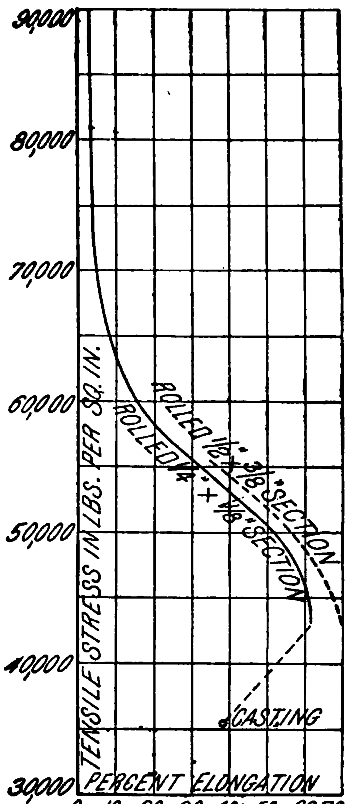


FIG. 467.—Showing the Relation between the Ultimate Strength and the Ultimate Elongation of Brass. (*Fr. Com. Rep.*, vol. III, Pl. V.)

and an elongation of nearly 40 per cent are given in Fig. 467. Thus in a ribbon $\frac{1}{4}$ in. × $\frac{1}{8}$ in. in cross-section the strength is 45,000 lbs. per square inch with 60 per cent elongation; 60,000 lbs. with 15 per cent elongation; and 90,000 lbs. with 3 per cent elongation.

393. The Strength of Bronze.—The ultimate tensile strength of all possible compositions of copper, zinc, and tin, in the form of unworked castings, is given in Fig. 468.* This is similar to that first published by Dr. Thurs-

* This is the same as Fig. 76, which is repeated here for convenience.

ton, but his was constructed from torsion tests, while this is made wholly from tension tests. It is the property of any equilateral triangle that the sum of the normals from any point in it to the three sides is equal to the common altitude of the triangle. Hence if these altitudes be each made to represent percentages, from zero to 100, and so graduated, each starting with 100 at the apex and reducing to zero at the opposite base, these may each represent a scale of one of the three ingredients, copper, zinc, and tin,

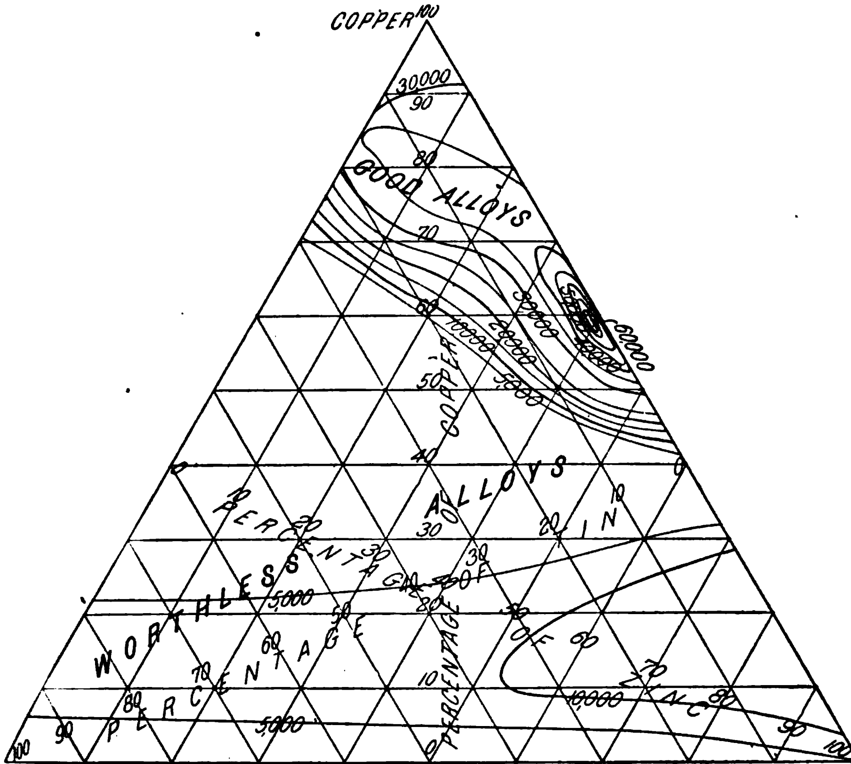


Fig. 468.—Showing the Tensile Strength, in Pounds per Sq. Inch, of All Possible Combinations of Copper, Tin, and Zinc, in the Form of Unrolled or Unforged Castings. (Compiled by the Author from the Records of the *U. S. Test Board* 1881.)

which go to make up all the bronzes. By drawing lines through these points of division in the altitudes, parallel to the bases, the triangle is subdivided into a series of similar smaller triangles as shown in Fig. 468. Any possible composition of copper, tin, and zinc, each represented as a certain percentage of the whole, may now be represented graphically by a location on this diagram. Its normal distances from each of the three sides, read off on the section lines in percentages, are at once the percentages of these three ingredients in that composition, the sum of these, of necessity, always being 100.

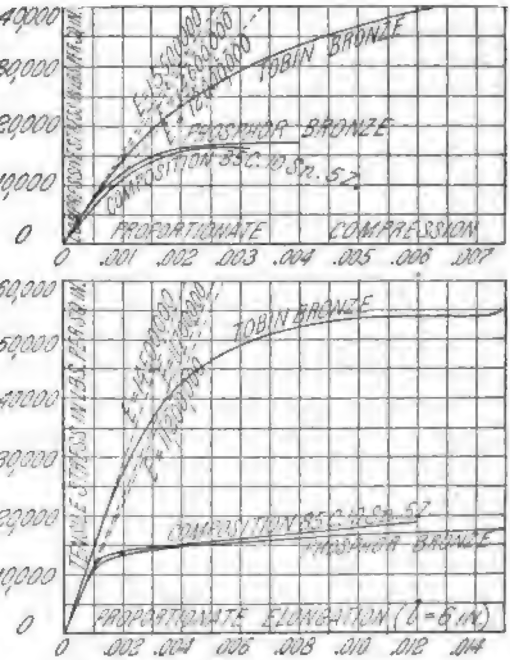


Fig. 469.—Results of Tension and Compression Tests on Three Alloys used for Valve-stems. Tobin bronze rolled, others plain castings. (Russell, *Jour. Assoc. Eng. Socs.*, vol. xv, p. 207. Tests made by the Author.)

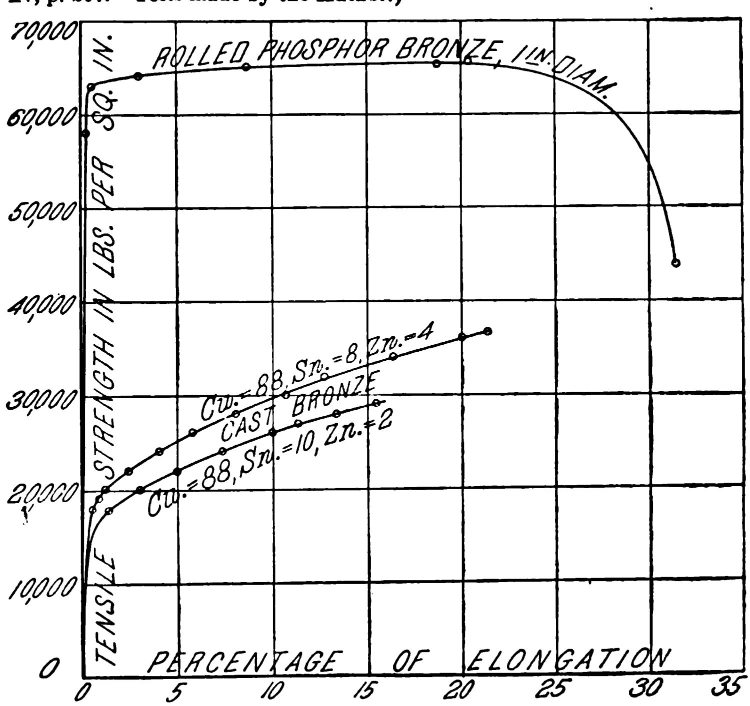


Fig. 470.—Tension Stress diagrams of Cast and Rolled Bronzes. (*Wat. Ars. Rep.* 1885.)

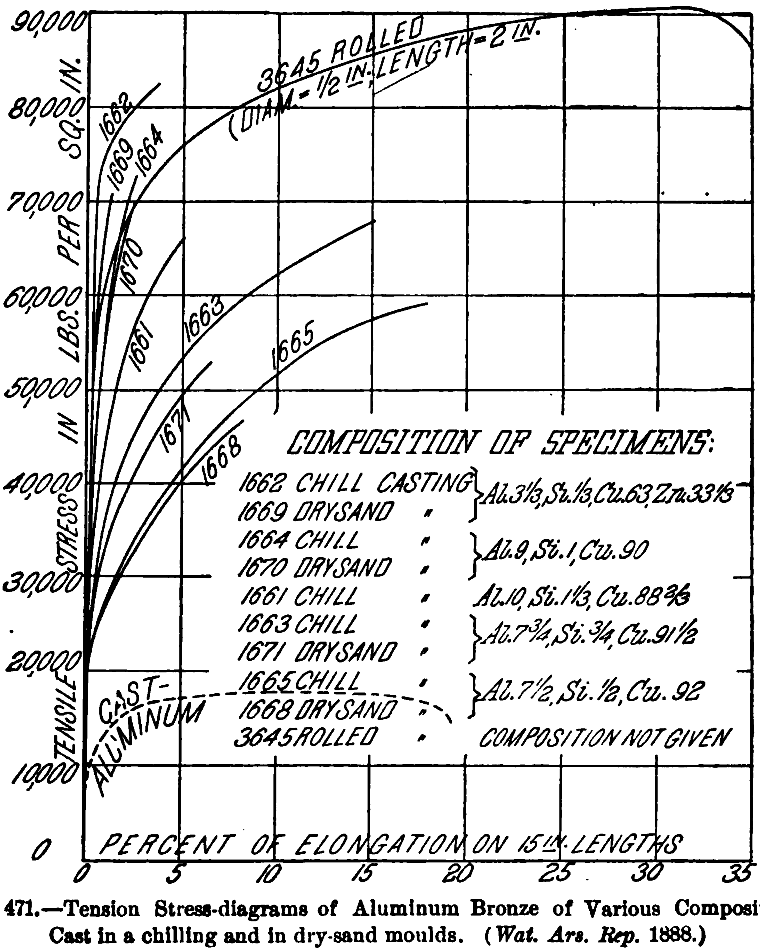


Fig. 471.—Tension Stress-diagrams of Aluminum Bronze of Various Compositions, Cast in a chilling and in dry-sand moulds. (*Wat. Ars. Rep.* 1888.)

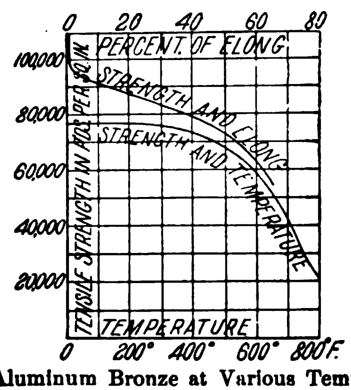


Fig. 472.—Strength of Aluminum Bronze at Various Temperatures and for Various Percentages of Elongation.

From an examination of this chart it is at once evident that only those alloys near the copper apex are of any value, the strongest being, however, near the copper-zinc side, where the composition is about 59 per cent copper, 39 per cent zinc, and 2 per cent tin. The tensile strength of such a casting, if properly made, is about 60,000 lbs. It is too brittle, however, to be of much value. The most valuable alloys are those having an ultimate strength of from 35,000 to 40,000 lbs. tensile strength, this having from 20 to 30 per cent elongation. This is found in the vicinity of 75 to 85 per cent copper, 17 to 5 per cent zinc, and 8 to 10 per cent tin.

Tobin Bronze is simply such a composition as the above hot-rolled after casting. The effect of this rolling is to greatly increase both the strength and the ductility, as shown by Fig. 469.* This material is in almost every respect similar to soft steel, so far as its mechanical qualities are concerned. It has the further advantage of not corroding under ordinary conditions, hence its extraordinary value as a structural material. The author has seen, however, some very remarkable and unexplained fractures of this material which leads him to suspect its reliability.

Phosphor-bronze has no special mechanical properties other than marks all good bronzes (see Fig. 470). Phosphorus is used to destroy the effects of oxidation in melting rather than to add to the strength or ductility otherwise. In destroying these oxides it does improve the product; but if the melting is performed in such a way as to prevent oxidation, there is no need of the phosphorus.

394. Aluminum-bronze may have great strength in both the cast and the rolled forms, as shown in Fig. 471, where many tests on different compositions and from different kinds of moulds are plotted from the same origin. The effect of the rolling in increasing the strength and the ductility is evident. A small percentage of aluminum is thus seen to greatly improve the bronze although its strength alone is very small, as is also shown in this figure.

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* The total elongation, which was over 30 per cent, is not shown in this figure.

CHAPTER XXIX.

THE EFFECTS OF TEMPERATURE ON THE MECHANICAL PROPERTIES OF METALS.

EFFECTS ON IRON AND STEEL.

395. As Shown by Stress-diagrams.—This subject has been very fully and carefully investigated at the testing laboratory of the U. S. Arsenal at Watertown, Mass., and a full series of diagrams, similar to that shown in

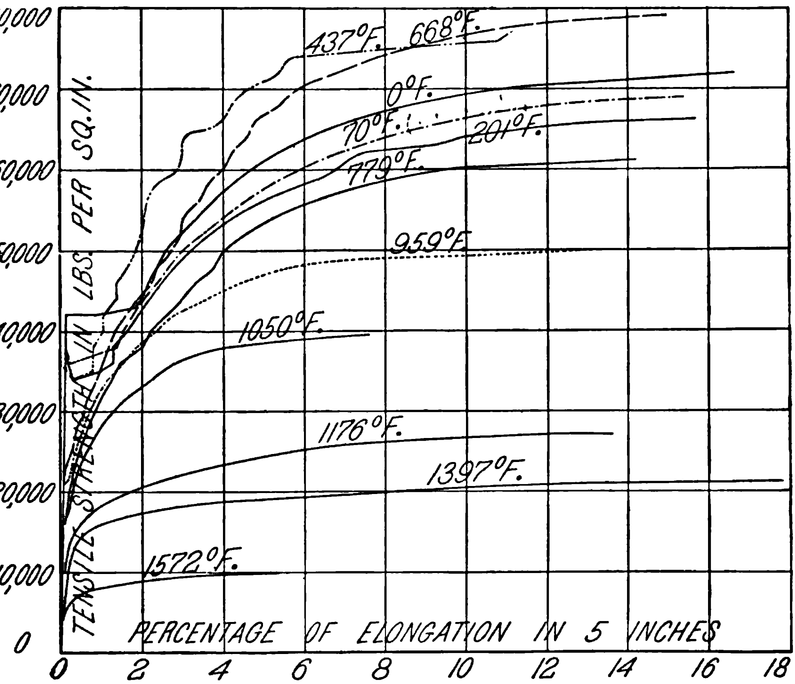

FIG. 473.—Stress-diagrams of Steel Bars, 0.20% Carbon, 0.45% Manganese, at Various Temperatures. (*Wat. Ars. Rep.* 1888.)

Fig. 473, is given in the report for 1888. The curves in this figure exhibit the action of 0.20 per cent carbon steel, having a normal tensile strength at 70° F. of 70,000 lbs. per square inch, with a normal elastic limit of some 37,000 bls. per square inch. Fig. 473 reveals both the elastic limit and the ultimate

strength, both of which are above the normal at 0° F., and below the normal at 210° F. The ultimate strength then increases with a rising temperature, reaching a maximum at about 600° F., from which temperature it regularly

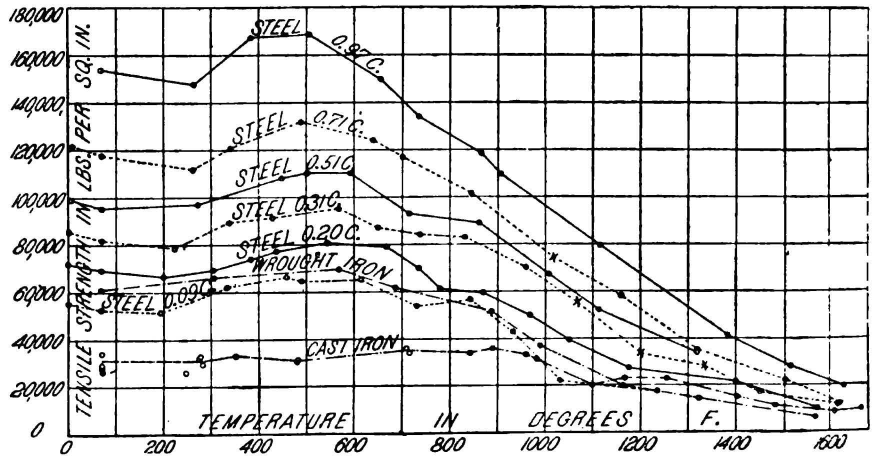

FIG. 474.—Variation of Tensile Strength with Temperature. (*Wat. Ars. Rep.* 1888.

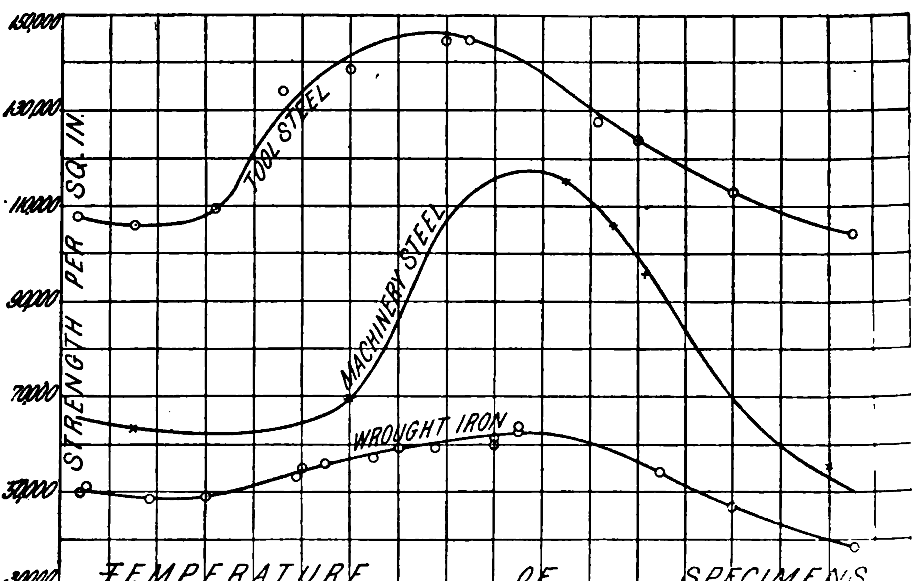

FIG. 475.—Variation of Tensile Strength of Wrought Iron and Steel for Varying Temperatures. Cornell University Tests. (*Jour. West. Soc. Engrs.*, vol. I.)

diminishes in ultimate strength, reaching 60,000 lbs. at 800° F., 50,000 lbs. at 960° F., 40,000 lbs. at 1050° F., 30,000 lbs. at 1150° F., 20,000 lbs. at 1400° F., and 10,000 lbs. at 1570° F. These simultaneous values are better

read off from Fig. 474 than from Fig. 473. In the former, also, are found the variations of the ultimate strength of all grades of steel, and of wrought and cast iron.

Returning now to Fig. 473, it will be seen that *the elastic limit regularly and continuously diminishes from the zero temperature*, where it is some 42,000 lbs. per square inch, the metal becoming regularly more plastic as the temperature rises. This is also shown in Fig. 478.

In Fig. 476 we see the relative effects of slow and rapid applications of

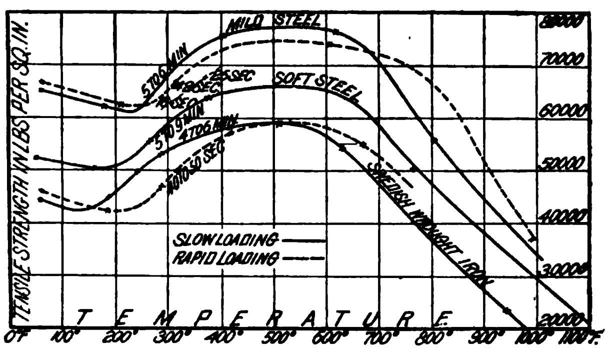

Fig. 476.—Ultimate Tensile Strength of Steel and Wrought Iron at Temperatures between Freezing and 1000° F. for Slow and Rapid Loading. (*Fr. Com. Rep.*, vol. II, Pl. XX.)

the load on wrought iron and steel at different temperatures. At ordinary temperatures the quick loading develops a greater ultimate tensile strength than the slow loading. Between 250° and 700° F. for steel, and between 150° and 500° F. for wrought iron, the quick loading gives a less ultimate strength, while beyond these higher temperatures the quick loading again gives the greater strength.

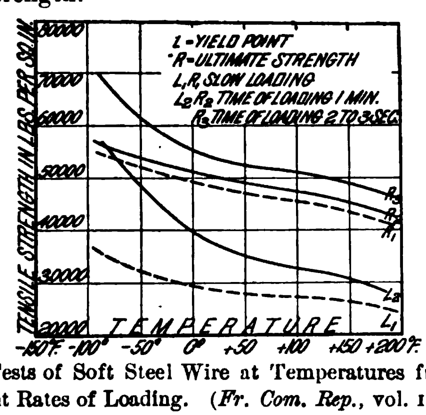

Fig. 477.—Tension Tests of Soft Steel Wire at Temperatures from − 90° to + 200° F. for Different Rates of Loading. (*Fr. Com. Rep.*, vol. II, Plate XX.)

Similar effects are shown in Fig. 477 for soft steel wire, for both the ultimate strength and the yield-point or apparent elastic limit, for temperatures between − 90° and + 200° F.

396. The Change in the Elastic Limit is by far the most important of all the changes produced by rising temperatures, so far as structural use is concerned. Commonly only the ultimate strength is given for rising temperatures, and as this increases up to 500° or 600° F., it is assumed that the working strength increases also. That this is not the case is shown for one grade of steel in Fig. 473, and for all grades of steel combined in Fig. 478. Here the " mean variation in the elastic limit " curve continuously descends

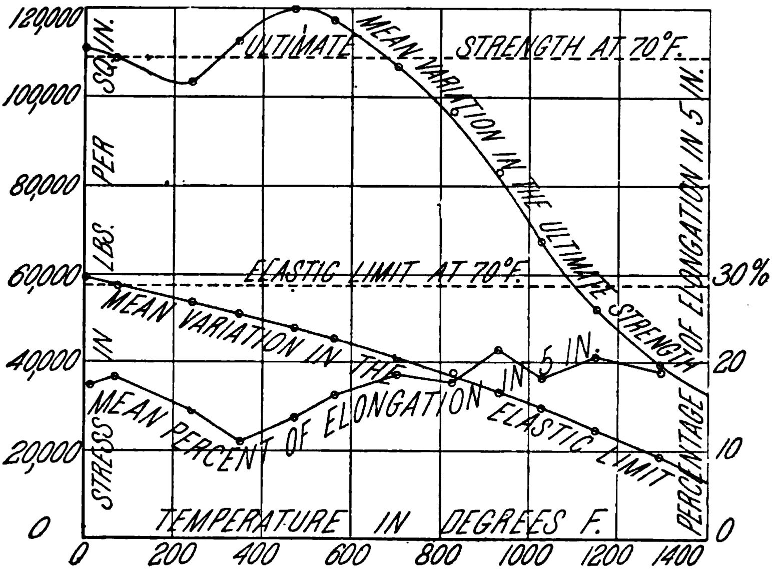

FIG. 478.—Grand Mean Curves from Temperature Tests on Steel Rods 0.8 in. in diameter, turned from 1¼-in. rods, of ten different degrees of hardness, from 0.09% to 0.97% C. (*Wat. Ars. Rep.* 1888, p. 245.)

from a zero temperature, the mean results falling almost exactly in a smooth curve, which is nearly a straight line, while the " mean ultimate strength " curve has a minimum point at 200° F. and a maximum point at 500° F., after which it regularly decreases also. Thus at 500° F. the mean ratio of elastic limit to ultimate strength, for all grades of steel, is only 0.36, while at ordinary temperatures, from zero to 100° F., it is 0.57, as shown by Fig. 478.

For structural purposes, therefore, the working strength of wrought iron and steel must be regarded as regularly diminishing, while the temperature increases, the rate of diminution being about 4 *per cent for each* 100° *F. increase in temperature.*

Similar curves in Fig. 479 do not indicate this uniform reduction from a zero temperature, but they are not based on as extensive a series of tests as those summarized in Fig. 478.

397. The Change in Ductility.—The great reduction in the elongation of wrought iron and steel, for temperatures from 100° to 400° F., with a

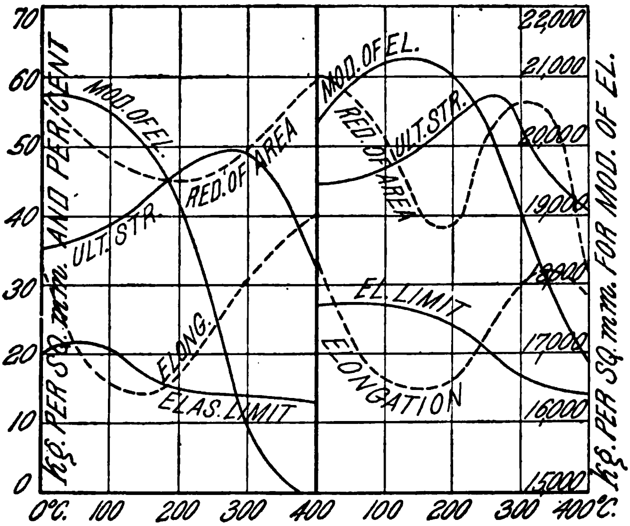

FIG. 479.—Tensile Properties of Wrought Iron and of Open-hearth Steel at Various Temperatures Centigrade. (*Berlin Testing Lab.* 1893.)

minimum at about 300° F., is a remarkable fact which could not have been predicted. Thus wrought iron with $22\frac{1}{2}$ per cent elongation at a temperature of 80° F. has but 7 per cent elongation at 300° F., as shown in Fig.

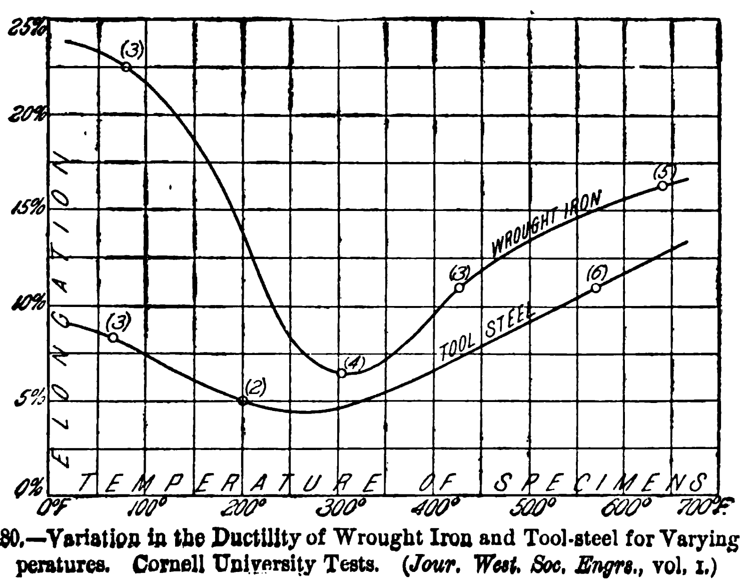

FIG. 480.—Variation in the Ductility of Wrought Iron and Tool-steel for Varying Temperatures. Cornell University Tests. (*Jour. West. Soc. Engrs.*, vol. I.)

480, while from Fig. 479 a 32-per-cent elongation of both wrought iron and mild steel at 32° F. reduces to 14 per cent at 300° F. Above this temperature the elongation increases again, reaching its normal amount at a temperature of some 600° F.

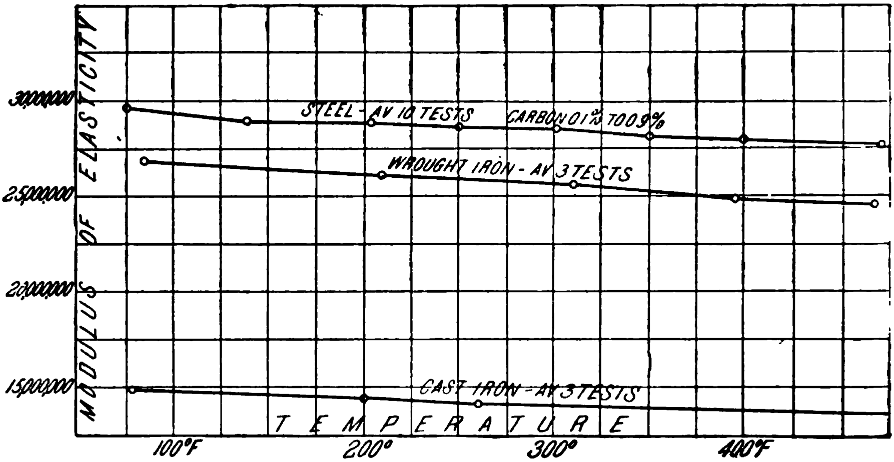

FIG. 481.—Effect of Moderate Temperatures on the Modulus of Elasticity. (*Wat. Ars. Rep.* 1887.)

398. The Change in the Modulus of Elasticity is shown in Figs. 479 and 481. In all cases it regularly decreases for rising temperatures, except that the Berlin tests on steel, Fig. 479, show a small increase in the modulus

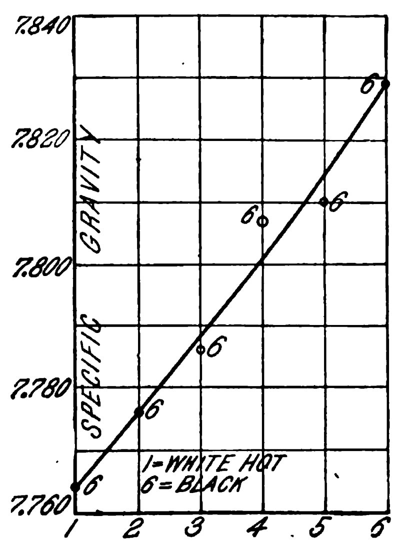

FIG. 482.—Variations in the Specific Gravity of Steel at Different Temperatures. Each point the mean of six observations. the carbon varying from 0.53% to 1.08%. (Langley, in *Am. Chem.*, 1876.)

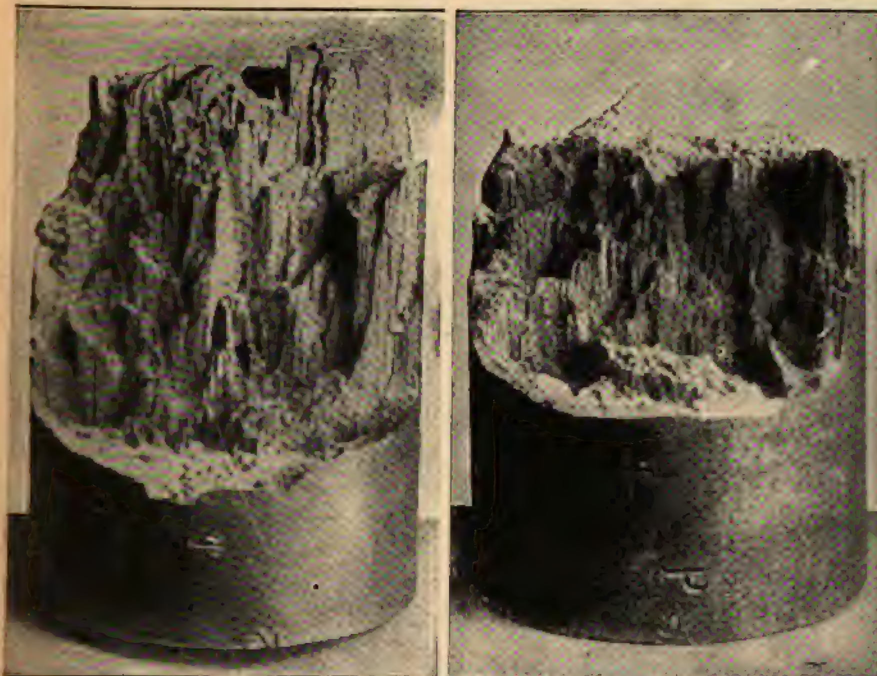

FIG. 483.—Hot Tests of Wrought-iron Car-axles. Temp. 300° F.

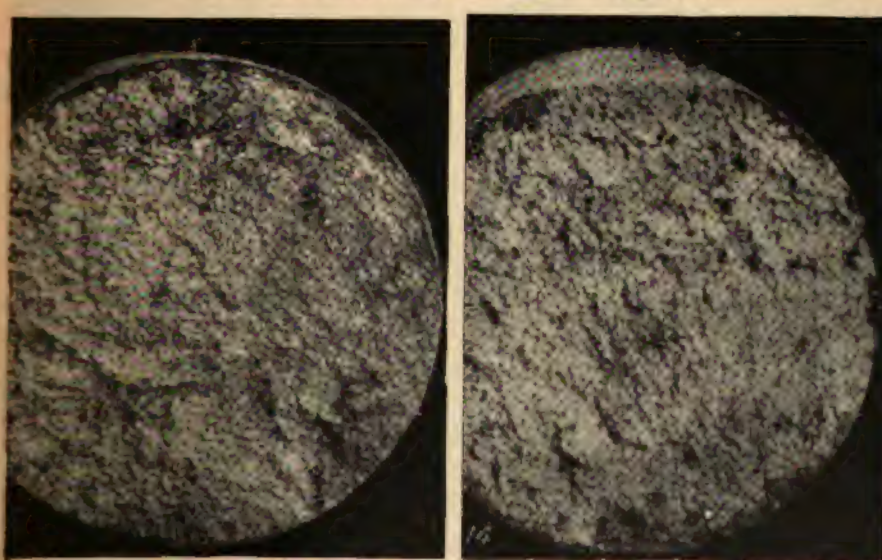

FIG. 484.—Cold Tests of Wrought-iron Car-axles. Temp. – 18° F.

Impact Tests of Car-axles, 4¾ in. in Diameter, showing Characteristic Fractures at 300° F. and at – 18° F. (Thos. Andrews, M. Inst. C. E., before the *Soc. Engrs.* (London), 1896, in a Bessemer Premium Paper.)

from freezing to 200° F. No such effect is shown in Fig. 481. In general it may be said that for wrought iron and steel the modulus of elasticity decreases about 2 per cent for each 100° F. increase in temperature.

399. Effect on Specific Gravity.—This is shown in Fig. 482 to be quite uniform, but no absolute temperatures were determined. We can only say that in cooling from a white heat to black the specific gravity increased from 7.76 to 7.83, an increase of nearly one per cent variation in temperature.*

400. Effect on Resistance to Impact.—This is shown, for wrought-iron car-axles, in Fig. 485, and the change in the fracture from a crystalline appearance at a temperature of − 18° F. to a fibrous appearance at 300° F. is well shown in Figs. 483 and 484. The minimum toughness is found at 300° C. or 570° F., which agrees substantially with the temperature of maximum ultimate tensile strength, but of minimum elongation. This could have been predicted from the reduced ductility at this temperature. The paper here cited contains a great many photographic reproductions of fractures from which those in Figs. 483 and 484 have been selected as characteristic. They are all very much alike.

Mr. Andrews's impact tests of wrought-iron car-axles at zero and at 100° F. show a great difference in the resistance to impact even for this

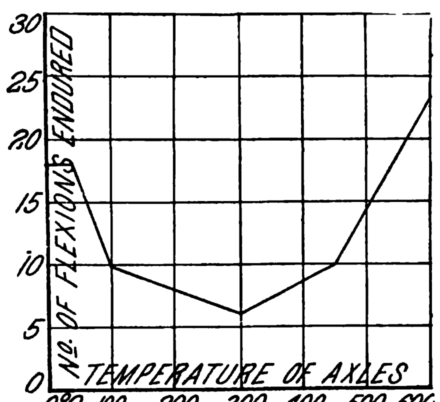

FIG. 485.—Endurance Tests of Wrought-iron Railway axles at Varying Temperatures Centigrade. Axles deflected by impact and then turned over and deflected in the opposite direction. (Andrews, before the *Soc. of Engrs.* (London), 1896.)

small variation in temperature.* These axles were 4½ in. in diameter and rested on supports 3½ ft. apart. They were tested by dropping a tup weighing 2240 lbs. a distance of 30 in., the axle being turned over after each blow and its temperature restored, until it ruptured.

These tests serve also to emphasize the fact that wrought iron is an extremely variable material when forged in large masses, and by no means as

* From Proc. Inst. Civ. Eng., vol. xciv. p. 209.

TABLE XXXVI.—ANDREWS' TESTS ON WROUGHT-IRON CAR-AXLES AT 0° AND 100° F.

Cold Tests at 0° F.			Warm Tests at 100° F.		
Number of Axle.	Sum of all Deflections of Axle in Inches.	Total Number of Blows causing Fracture.	Number of Axle.	Sum of all Deflections of Axle in Inches.	Total Number of Blows causing Fracture.
44	4.8	8	45	13.3	23
46	5.7	8	47	11.9	15
48	0.8	2	49	19 9	23
50	6.6	8	51	14.4	17
52	8.7	11	53	21 7	22
54	38.6	44	57	71.1	107
55	4.5	6	63	9.1	12
56	6.9	10	64	31 4	49
58	7.2	9	65	32.1	44
59	5.3	7	67	40.9	54
60	4.0	6	69	17.9	24
61	10.3	14	70	16.4	22
62	5.6	8	71	47.5	66
66	25.7	33	72	43.8	62
68	26.2	32	73	37.5	57
77	21.6	29	74	25.9	34
78	66.0	84	75	12.1	16
79	58.8	76	76	17.2	25
80	49.4	64	81	17.8	22
83	25.9	34	82	23.4	35
84	30.1	42	89	24.5	32
87	25.3	32	85	34.4	35
88	3.4	5	86	10.6	56
90	16.1	20	111	34.6	53
91	35.4	48	98	52.8	78
92	8.6	12	113	30.4	45
93	7.4	10	120	23.1	32
94	3.4	5	103	34.5	49
95	3.0	5	121	25.9	40
96	31.2	43	108	41.1	54
Average	18.2	23.8	Average	27.7	37.1

uniform as mild steel. If steel axles would show one half as great a range in results as is here revealed for wrought iron, they would all be rejected without any hesitation.

EFFECTS ON COPPER AND BRONZE.

401. Effects on Copper.—These are shown in Fig. 486. Both the elastic and the ultimate strength regularly diminish for rising temperatures, while the elongation remains nearly constant up to 600° F. The modulus of elasticity rises to a maximum at the boiling temperature, where it is 15 per cent higher than at a freezing temperature, and then rapidly declines. The elastic-limit strength of rolled copper may be said to diminish at the rate of 5 per cent per 100° F. increase in temperature.

402. Effects on Bronze.—The elastic strength, the ultimate strength, and the ductility of bronze are but little affected by rising temperatures up to 600° F., the reduction in strength being only about 2 per cent per 100° F.

FIG. 486.—Tensile Properties of Copper and Delta-metal at Various Temperatures Centigrade. (*Berlin Testing Lab.*, 1898.)

within this limit, as shown in by Fig. 487. The modulus of elasticity rises some 20 per cent at 550° F., and then rapidly falls.

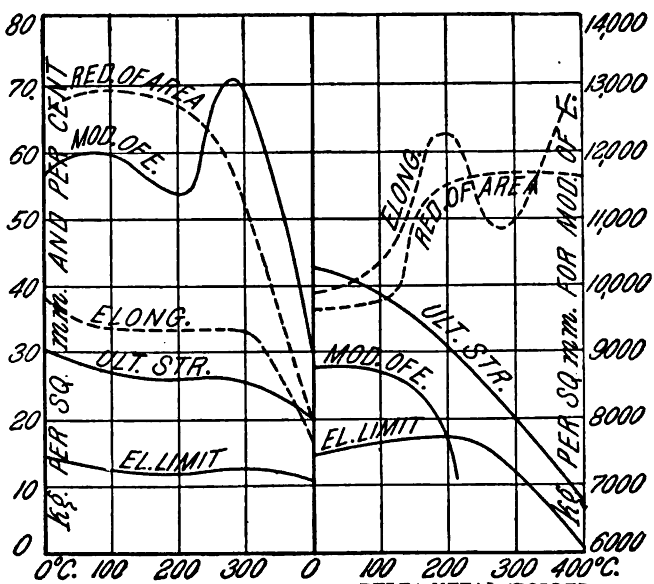

FIG. 487.—Tensile Properties of Manganese Bronze and Delta-metal (rolled) at Various Temperatures Centigrade. (*Berlin Testing Lab.*, 1898.)

403. Effects on Delta-metal.—These are shown in Figs. 486 and 487 for cast and rolled delta-metal respectively. The ultimate strength falls much more rapidly than the elastic limit, this remaining nearly constant up to about 500° F. The modulus of elasticity falls rapidly after passing 300° F., but the ductility increases to 400° F. for rolled, and to 550° F. with the cast, metal, the elongation being 60 per cent and 55 per cent respectively at these limits.

404. Conclusions.—In general it may be said that copper and its useful alloys have their mechanical properties changed but little for the variations of temperature ordinarily occurring in the use of these materials in the arts. See Fig. 472 for aluminum bronze.

For very low temperatures the static strength of iron and steel increases somewhat, but both elastic limit and elongation, or ductility, decrease, so that the resistance to shock is considerably reduced. The bad effect of cold weather, therefore, is shown on materials subjected to heavy blows, like railroad rails. It is not practicable to make shock tests at temperatures lower than that found out-of-doors in winter, or such as may be created in a large refrigerating warehouse. Tests at extremely low temperatures, therefore, such as shown in Fig. 477, are necessarily limited to tension tests of small specimens, which can be surrounded by a cooling apparatus.

If shock tests are made on artificially cooled bars, at ordinary temperatures, they should be returned to the refrigerator after each blow, or at most after each second blow.

The curves shown in Figs. 474 to 480 show that in tension tests at ordinary atmospheric temperatures no note need be made of the particular temperature of the test bar. It is very different, however, with impact tests as shown for wrought iron in Fig. 485. Here even these atmospheric variations are important and the temperature should always be noted. In order to make such tests comparable they should be made at about the same temperature, and 60° to 70° F. has been selected as the standard. Most kinds of test-specimens could be brought to this temperature by immersion in water.

CHAPTER XXX.

RESULTS OF TESTS ON CEMENTS, CEMENT-MORTARS, AND CONCRETES.

TENSILE AND COMPRESSIVE STRENGTH OF CEMENTS AND CEMENT-MORTARS.

405. Tensile Strength of Natural Cement.—As shown in Art. 315, Fig. 337, the tensile strength of cement is a true index of its compressive strength. It was also stated in Art. 155 that the American natural cements are, as a class, of a superior grade, and that they are quite sufficient in strength for nearly all purposes for which cement is required. Occasional failures of this class of cements has, however, developed an undue popular prejudice against them. If reasonable precautions were exercised in testing such cements, a great deal of money could be saved with no prejudice to the works on which it might be used.

Fig. 489 contains the average results of many thousands of tests of

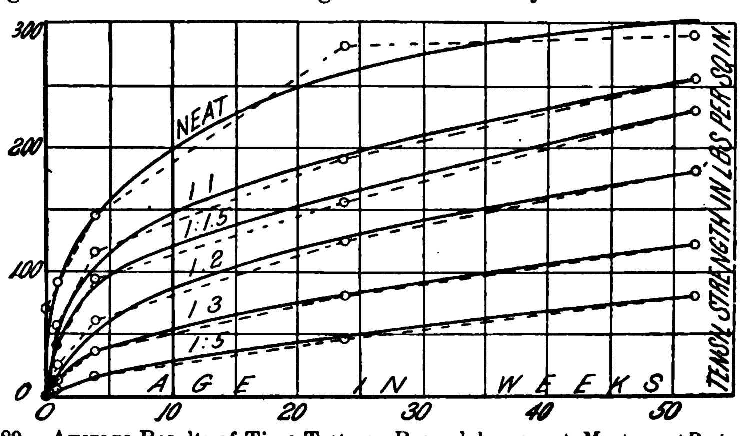

FIG. 489.—Average Results of Time Tests on Rosendale cement Mortar. (*Boston Main Drainage*, 1885, p. 121.)

Rosendale-cement mortars, extending over the several years of the construction of the Boston Main Drainage works. The usual mixture for natural-cement mortar is 1 C. : 2 S., and these tests give for this mortar an average tensile strength of 180 lbs. per square inch at the end of one year. In Fig.

490 this mixture had, in tests made during the construction of the Cairo bridge across the Ohio River, for Milwaukee cement, 160 lbs.; for Utica cement, 145 lbs.; and for Louisville, 140 lbs., this strength having been reached in each instance at the end of three months.

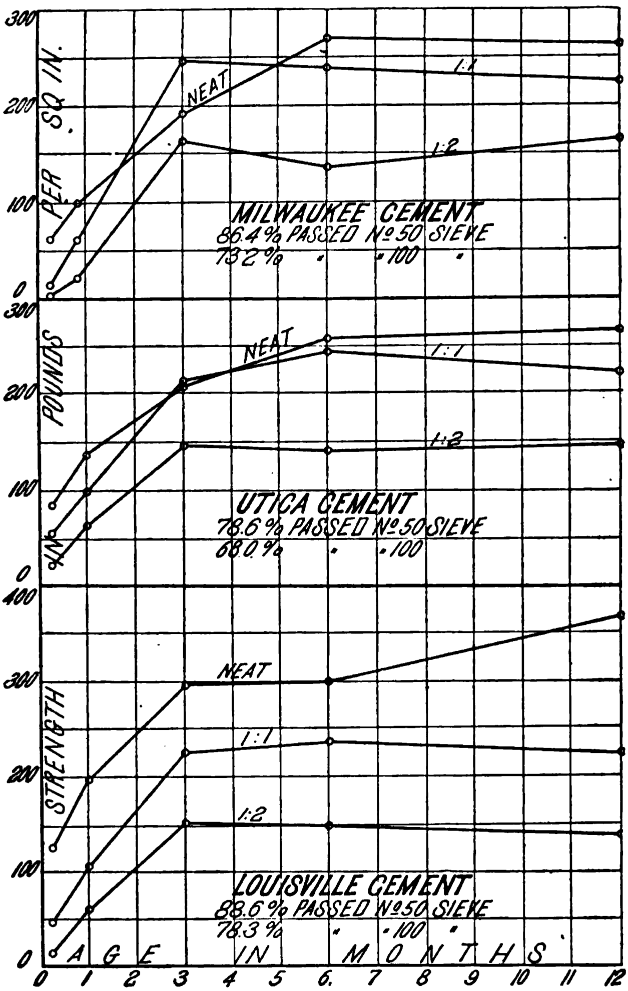

FIG. 490.—Time Tests on Three Standard Natural-cement Mortars. (*Jour. Assoc. Eng. Socs.*, vol. IX.)

Long-time tests of Louisville-cement mortar mixed the same as is usual with Portland cement, 1 C. : 3 S., gave at one year an average tensile strength of 230 lbs. per square inch, as shown by Fig. 491. This greater strength is probably due to the superior methods of making the test briquettes which have been followed in this department for many years. This

figure shows the average strength of neat Louisville cement to be 500 lbs. per square inch in one year when mixed on the "gig," a kind of "milk-shake" apparatus, described in *Engineering News*, vol. xxv. p. 3 (Jan. 3, 1891).

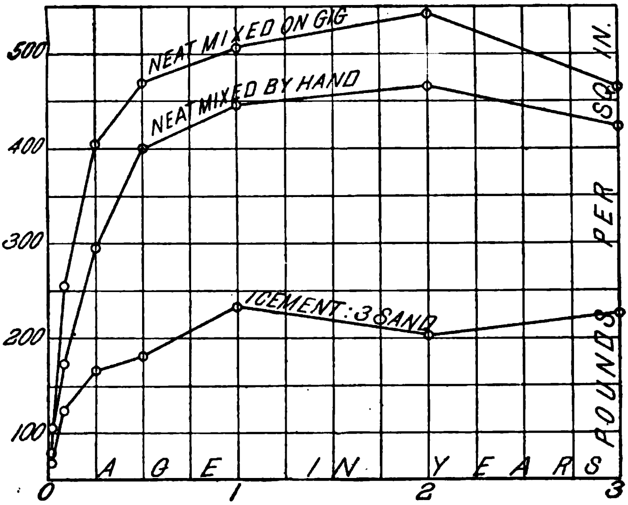

Fig. 491.—Average Results of Time Tests on Eight Brands of Louisville Cement. (*St. Louis Water-works*, 1896.)

Similar results have been obtained in the tests of natural cement made in connection with the building of the new Sault Ste. Marie Canal lock, as shown in Fig. 492. Here one brand of natural-cement mortars gave at one

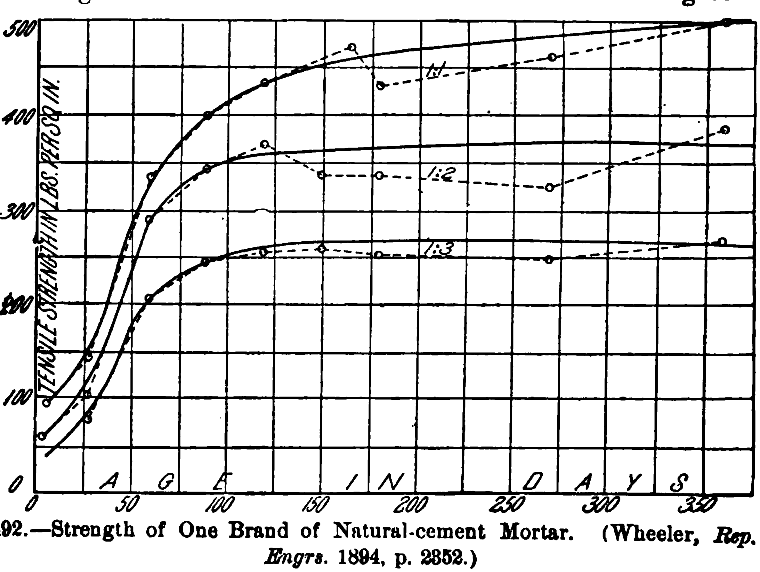

Fig. 492.—Strength of One Brand of Natural-cement Mortar. (Wheeler, *Rep. Chf. Engrs.* 1894, p. 2352.)

year, for 1 C. : 1 S., 500 lbs.; 1 C. : 2 S., 370 lbs.; and 1 C. : 3 S., 260 lbs. tensile strength. An average of five brands of natural-cement mortar, 1 C. : 1 S. gave at three months 380 lbs., and an average of ten brands of

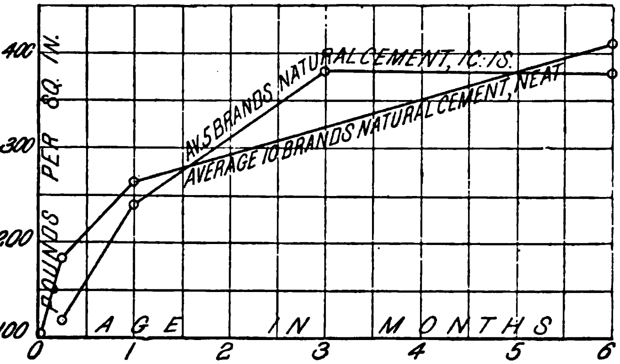

FIG. 493.—Strength of Natural Cement Neat and 1 C. : 1 S. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2983.)

natural cement neat gave at six months 410 lbs. tensile strength, as shown in Fig. 493.

These results all go to show that if reasonable care be exercised in

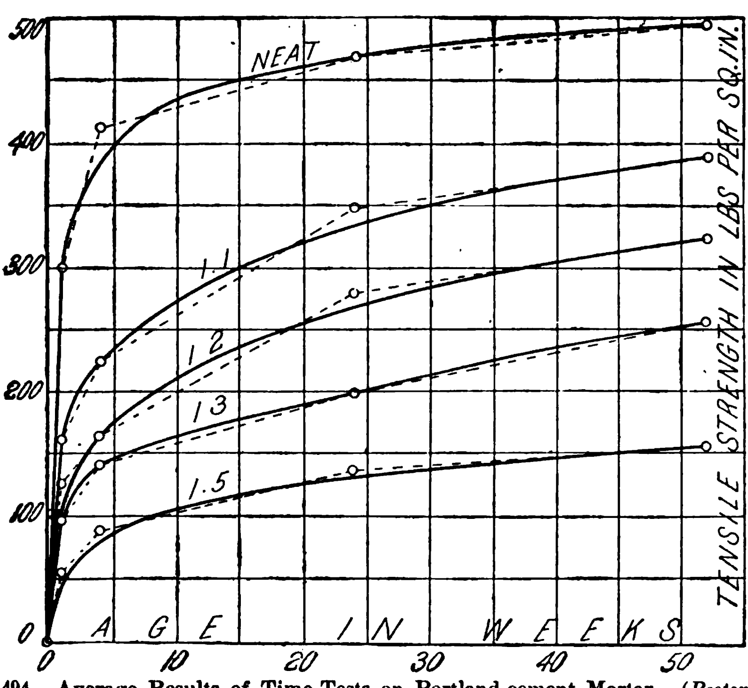

FIG. 494.—Average Results of Time Tests on Portland-cement Mortar. (*Boston Main Drainage*, 1885.)

inspecting and testing the cement, the standard American natural cements are abundantly strong for a large proportion of the work requiring the use of such material.

406. Tensile Strength of Portland Cement.—The average results of tests on Portland cement made on the Boston Main Drainage works are given in Fig. 494. Comparing this figure with Fig. 489, we may say that 4 to 1 of Portland cement is fully equal to 2 to 1 mortar of natural cement. Similarly, from these same figures we may say that neat Portland cement at one year

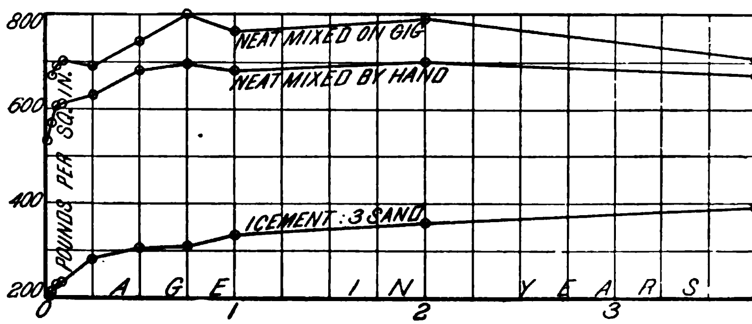

FIG. 495.—Average Tensile Strength of Fifteen Brands of Portland Cement. (*St. Louis Water Dept.*, 1896.)

is 60 per cent stronger than neat natural cement, and that standard mortar composed of 3 S. : 1 C. is nearly twice as strong when made of Portland

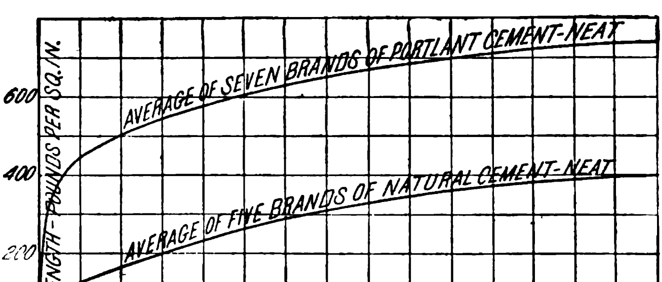

FIG. 496.—Results of Cement Tests made at the Iowa State University.

cement as when made of natural cement. Similar results on neat cement are shown in Fig. 496.

In Fig. 497 are shown Tetmajer's average relations of the strength of standard natural- and Portland-cement mortars at various ages up to one year, as percentages of the strength at 28 days. From this it appears that natural-cement mortar at one year is twice as strong as it is at 28 days, while Portland-cement mortar at one year is only 50 per cent stronger than at 28 days. Furthermore, the strength of the natural cement is still increasing, while that of the Portland cement has about reached its maximum.

In Fig. 498 are given the average results of tests on nine brands of Belgian Portland cement and on twelve brands of English Portland, both neat and 1 C. : 3 S. It would seem these mortar results are too low in

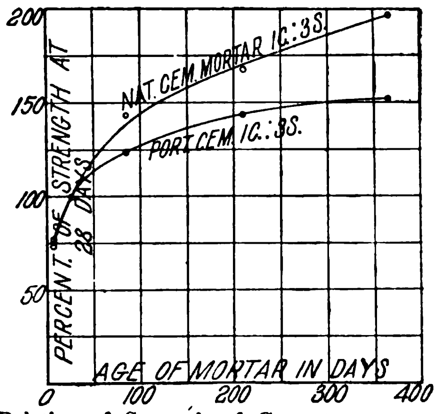

FIG. 497.—Average Relation of Strength of Cement-mortars which have Hardened under Water for periods less than One Year to the Strength of Twenty-eight Days. (Tetmajer's *Communications*, vol. VI. pp. 379–389.)

comparison with the results on the neat cement. In general standard mortar, 1 C. : 3 S., should reach one half the tensile strength of the neat cement at the end of a year. The strength of Portland cement, both neat and with

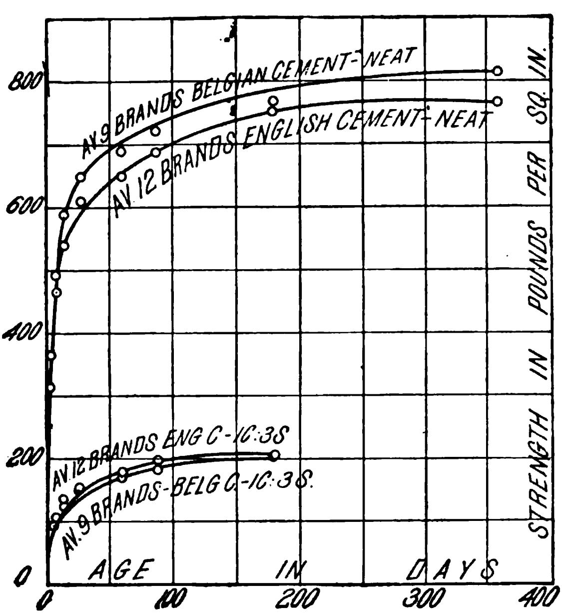

FIG. 498.—Average Results of Tension Tests on Belgian and English Portland Cements. (Allison, *Trans. Can. Soc. C. E.*, vol. IX, 1895, p. 296.)

sand, continues to increase for many years, as appears from Fig. 499. Here the strength of the standard mortar, 1 C. : 3 S., was 65 per cent of that of the neat cement at one year, while at five years it had increased to 75 per cent of the strength of the neat cement at that age. Short-time tests on

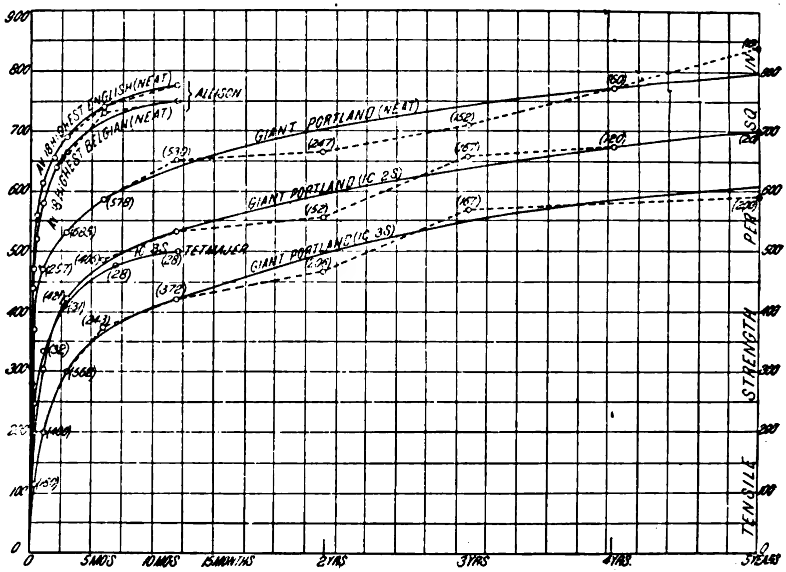

Fig. 499.—Long-time Tests of American and Foreign Portland Cements. Figures give number of tests averaged. (*Jour. Assoc. Eng. Socs.*, vol. xv. p. 193, and *Can. Soc. C. E.*, vol. IX.)

sand mixtures always give a much lower ratio of strength to that of the same cement neat than long-time tests. It will be observed, also, that the five-year tests in Fig. 499 were on an American cement. There is now no question as to the superior quality of many brands of American Portland cement. This is also shown by Fig. 500. In this figure the ratio of the strength of the mortar is so low as to lead to the conclusion that no special pains were taken to compact the briquettes. The strength of cement-mortar, 1 C. : 3 S., can readily be increased 100 per cent by mixing somewhat dry and using the Böhme hammer, Fig. 352, as compared to the strength of soft mortar which is merely pressed into the moulds. It is for this reason that American engineers adhere so uniformly to the neat test, the strength of neat briquettes not being so much affected by the method used for filling the moulds.

The relative effects of hardening in air and in water are shown in Fig. 502 for one brand each of natural, slag, and Portland cement. Evidently the continued presence of water is essential to the greatest strength of the Portland cement, while the natural cement reached a higher strength in the air.

A mixture of natural and Portland cement varies in strength between that of the true ingredients, strictly in proportion to the percentage of each used, as shown by Fig. 503.

A mixture of 1 S. : 1 C., if the mixing be very thorough, has about the same tensile strength as the same cement neat. (See Fig. 490.)

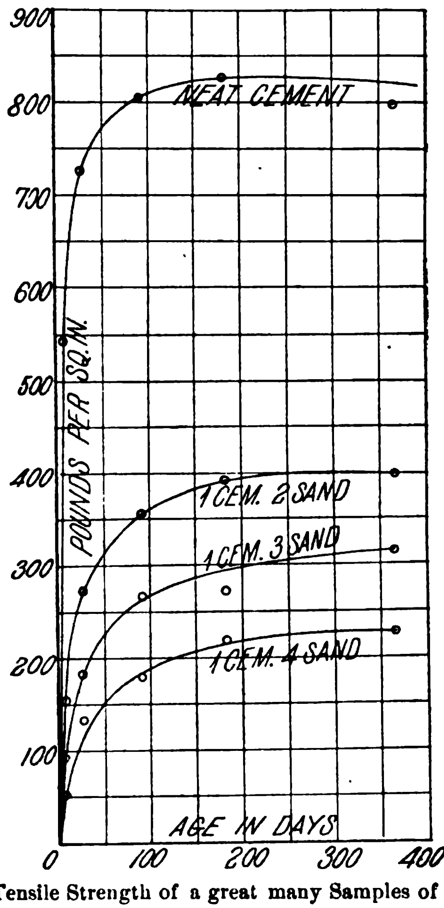

FIG. 500.—Average Tensile Strength of a great many Samples of One Brand of American Portland Cement (1896). (Robt. W. Hunt & Co.)

407. The Modulus of Elasticity of Portland-cement Mortars.—These are shown in Fig. 504 for two brands of Portland cement and one brand of slag-cement. The modulus increases with age, as would be expected, but it is also one third greater for standard mortar, 1 C. : 3 S., than it is for the neat cement, which could hardly have been predicted, especially when determined from a cross-bending test. The modulus is very much lower for the slag-cement than for the Portland cement; in other words, the slag-cement is more elastic, or resilient, than the Portland. This is an important quality

which should be further studied. The moduli of elasticity in compression of neat-cement mortars, and concretes are given in Art. 418, Figs. 546, 547, and 548.

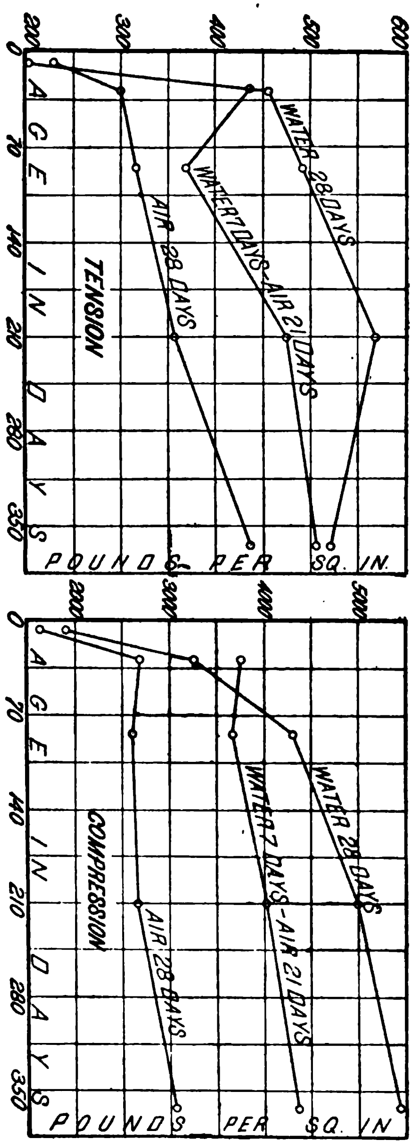

Fig. 501.—Effect of Air-hardening on one kind of Slag-cement (remaining portion of period in water). (Tetmajer, vol. VII. p. 136.)

408. Strength of "Sand-cement" Mortars.—Within a few years a new product has been introduced (from Denmark), composed of Portland cement reground with sand. This is a pure dilution, but it also makes available, by regrinding, the coarser particles of the cement, so that the new mixture may

now be mixed again with raw sand, the same as the pure cement, and the result is a considerable cheapening of the product for a given final strength. Thus if three parts of sand be ground with one part of Portland cement, the

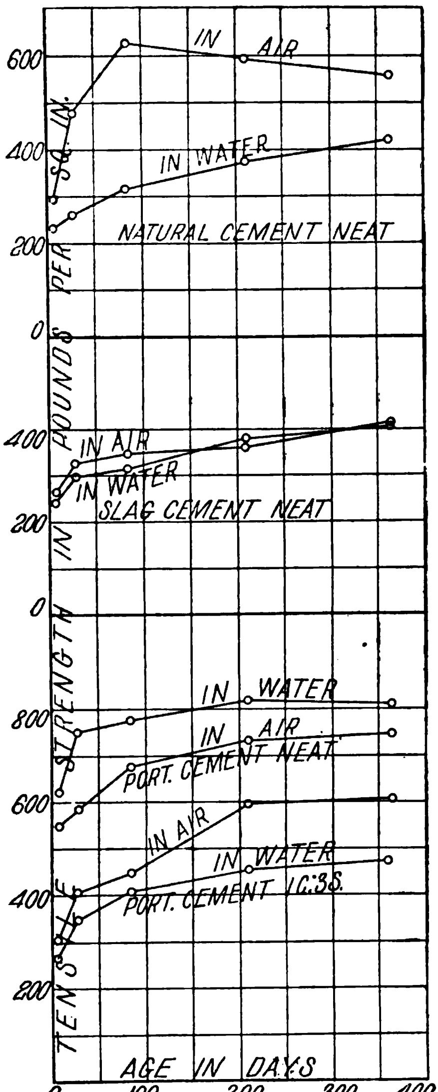

FIG. 502.—Relative Strength of Cement-mortars when Hardened in Air and in Water. (Tetmajer's *Communications*, vol. VI.)

product is four parts of "sand-cement." If now this be incorporated with raw sand in the proportion 1 : 3, we shall have to use 12 parts of raw sand,

making in all 15 parts of sand to 1 of cement. The formula for this mortar would be, therefore, 1 : 3 : 12 = 1 C. : 15 S. The tensile strength of

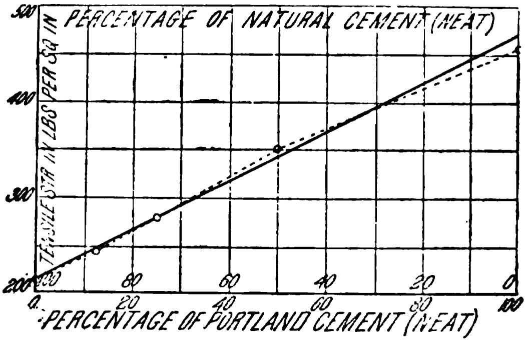

Fig. 503.—Strength of Mixtures of Natural and Portland Cement. Average of tests from one week to one year. (Wheeler, *Rep. Chf. Engrs.* 1894, p. 2350.)

various such mixtures, at ages up to one year, are given in Fig. 505, in comparison with the strength of standard mortar 1 C. : 3 S. Thus the mixture 1 : 2 : 6 = 1 C. : 8 S. gives almost as great strength as the ordinary 1 C. : 3 S.

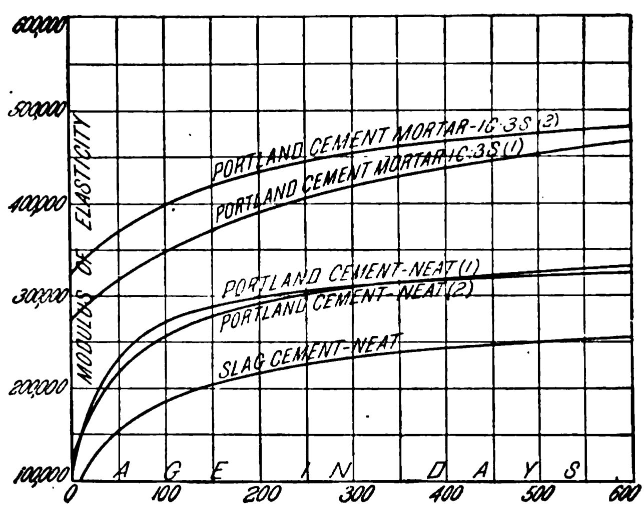

Fig. 504.—Modulus of Elasticity of Portland-cement Mortar as determined by Cross-bending Tests. (*Inst. Civ. Engrs.*, vol. cxi. p. 109.)

In Fig. 506 both the tensile and the compressive strengths of sand-cement mixtures are given from 1 C. : 3 S. to 1 : 3 : 8 = 1 C. : 35 S. In Fig. 507 these same results are plotted to the argument *cement ÷ total sand and cement.*

In Fig. 508 the tensile and the compressive strengths are given for a constant total ratio of sand to cement, but with different ratios of ground to unground sand.

While "sand-cement" is now (1897) manufactured at New York City, it is doubtful if it ever comes to be used very much in America, since it

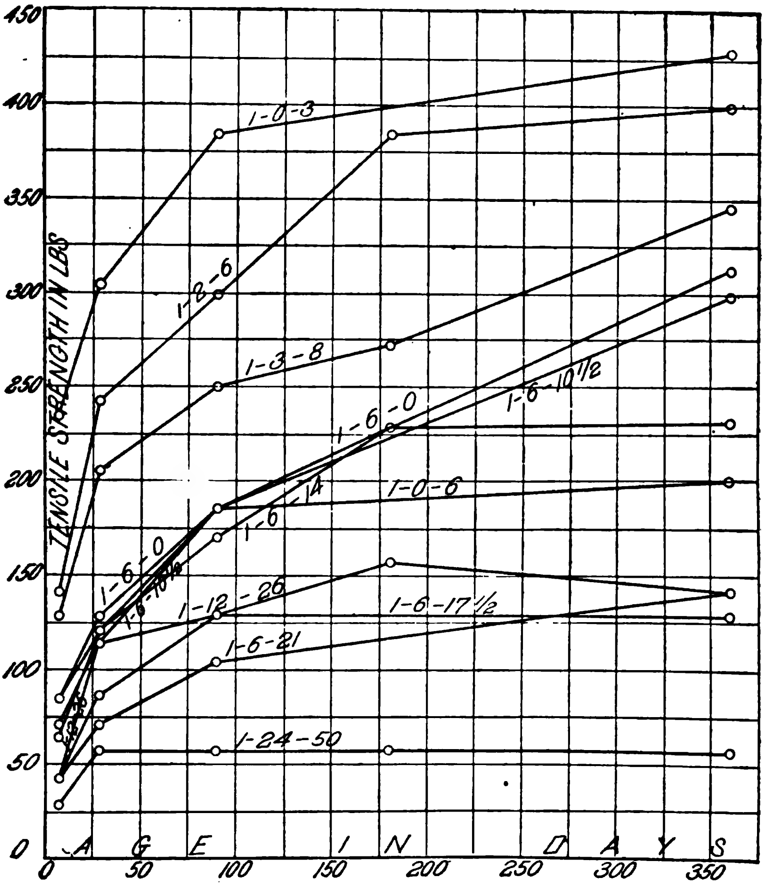

FIG. 505.—Strength of Sand-cement Mortar. The first figure denotes parts of Portland cement; the second, parts of ground sand; the third, parts of unground sand. (*Engr. News*, April 16, 1896, vol. xxxv. p. 254.)

cannot compete in price with our excellent natural cements, and because Portland cement will soon be made here in sufficient quantities to meet the entire home demand, and at prices so low that there will probably be little demand for this kind of dilution.

409. Variation of Strength of Cement-mortar with Increasing Proportions of Sand.—This law is largely a function of the methods employed in mixing the mortar and in compacting it in the moulds. When this is

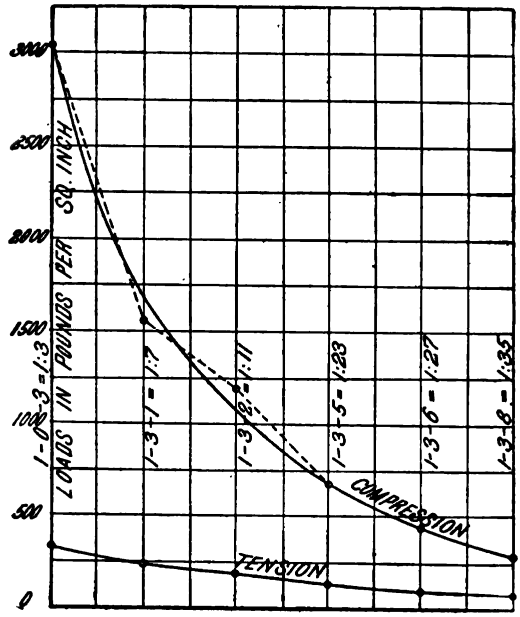

Fig. 506.—Strength of "Sand-cement" Mortar at Twenty-eight Days for Increasing Proportions of Free Sand when mixed with "Sand-cement" containing 1 C. : 3 S. (*Thonindustrie-Zeitung*, January 6, 1896.)

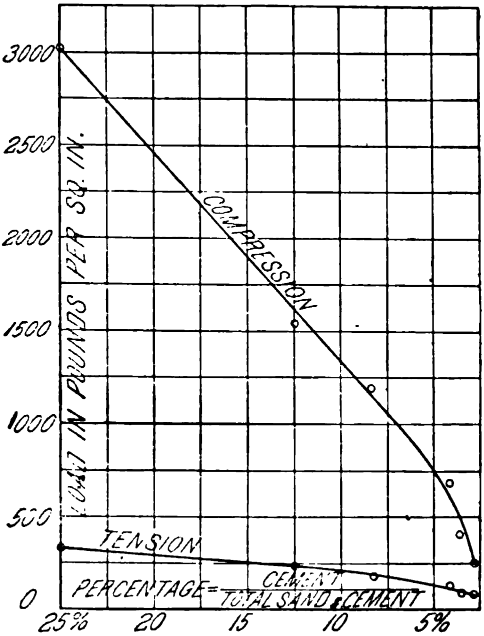

Fig. 507.—Strength of Sand-cement Mortar with Varying Proportions of Sand. (*Thonindustrie-Zeitung*, January 6, 1896.)

thoroughly done, the strength of mortar composed of 1 C. : 1 S. will be found to be about as strong as, and often stronger than, that of the neat

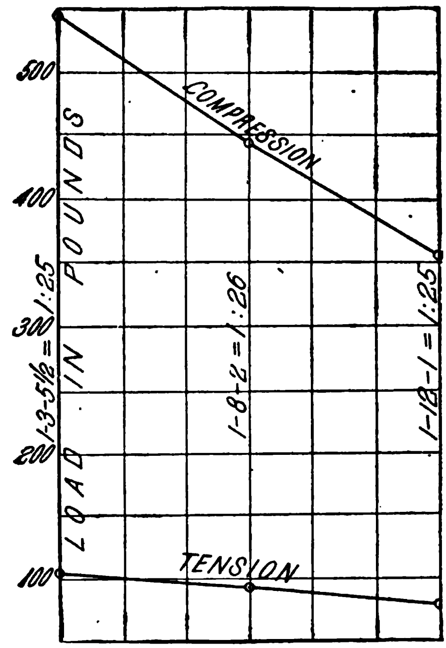

FIG. 508.—Variation in Strength of "Sand-cement" Mortar when the Total Proportion of Sand is constant, but a Varying Proportion of Ground and Unground. (*Thon-industrie-Zeitung,* January 6, 1896.)

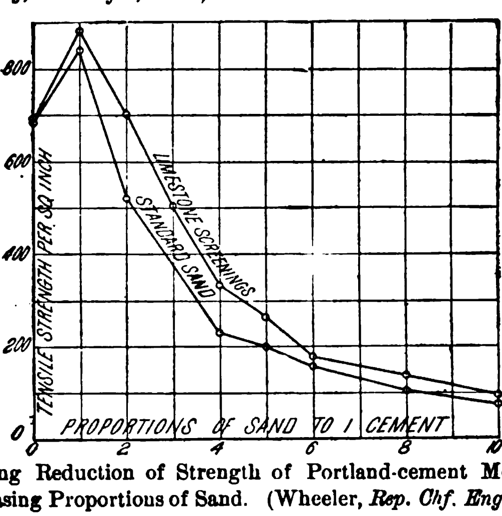

FIG. 509.—Showing Reduction of Strength of Portland-cement Mortar, Six Months Old, with Increasing Proportions of Sand. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2982.)

cement. Thus in ~~Fig. 517~~ the 1 : 1 mortar was stronger than the neat Portland cement. The standard mixture of 3 S. : 1 C. should have, in

general, abont one half the strength of the neat cement when six months old. It also appears from Figs. 509 and 511 that Portland-cement mortar

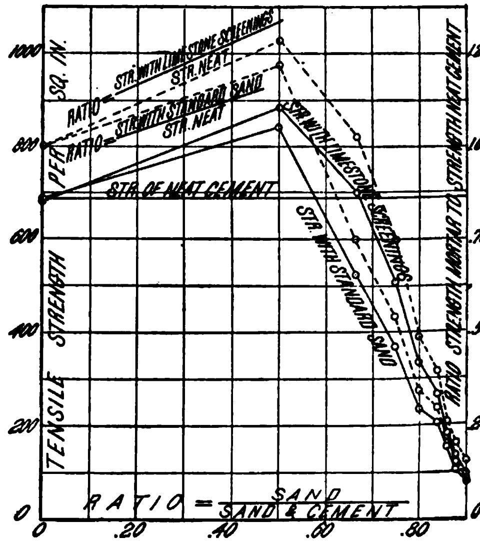

Fig. 510.—Showing Reduction in Strength of Portland-cement Mortars, Six Months Old, with Increasing Proportions of Sand. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2982.)

of 4 S. : 1 C. has the same strength at six months as natural-cement mortar of 2 S. : 1 C. of same age.

Similar relations appear in Figs. 489, 494, 512, 513, and 515.

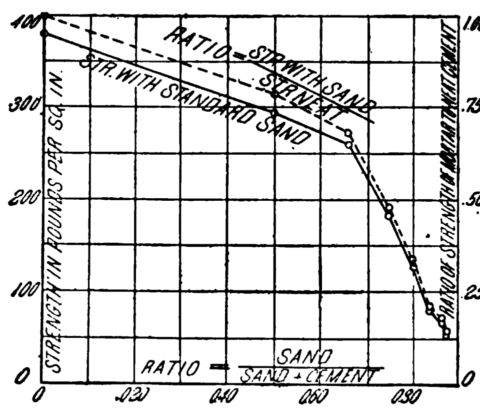

Fig. 511.—Showing Reduction of Strength of Natural-cement Mortar, Six Months Old, for Increasing Proportions of Sand. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2982.)

410. Variation of the Strength of Cement-mortars with a Variation in Size of the Sand-grains.—Photographs of sand-grains, natural size, obtained

by the use of graded sieves, are shown in Figs. 518 to 523. The effect of the variation in size of the sand-grains was discussed and results shown in

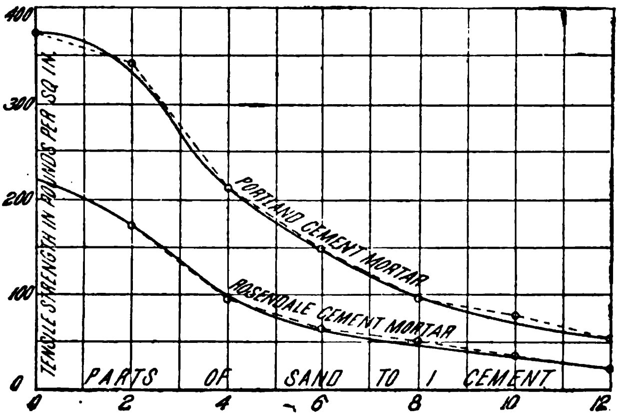

FIG. 512.—Strength of Cement-mortars, at Six Months Old, for Varying Proportions of Sand. (*Boston Main Drainage*, 1885, p. 122.)

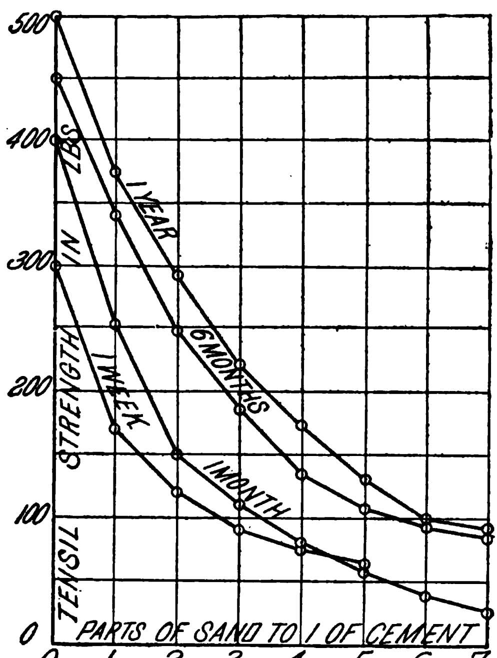

FIG. 513.—Tensile Strength of Portland-cement Mortar. (Baker, in *Masonry Construction*, p. 90.)

Art. 317. It there appeared that sand which passes a No. 20 and stops on a No. 30 sieve (20 and 30 meshes per linear inch) gave the strongest mortars.

It is shown in Fig. 516 that mortar from this grade of sand is from 25 to 50 per cent stronger than mortar made from sand which had passed a No. 40 and stopped on a No. 80 sieve.

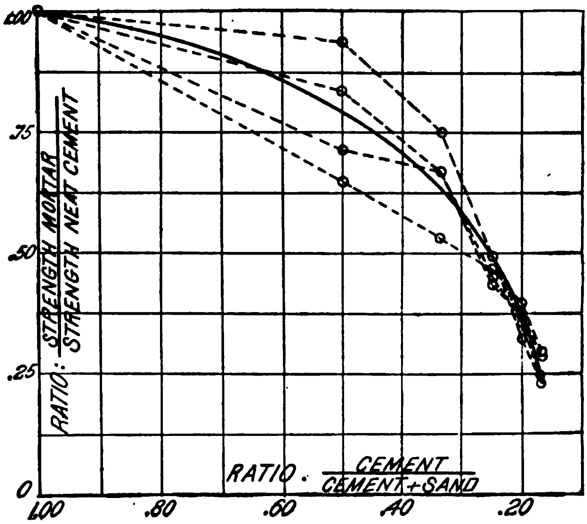

Fig. 514.—Ratio of Strength of Mortar to Strength of Neat Cement for Different Proportions of Sand. (*Rep. N. Y. State Engr.* 1894, p. 386.)

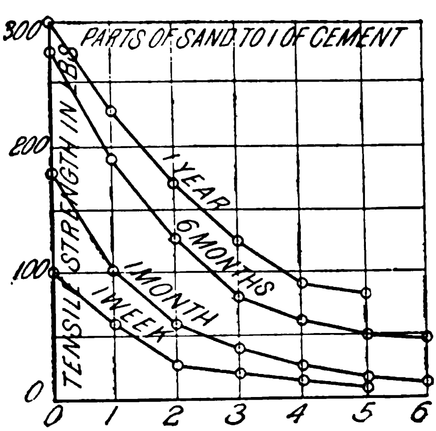

Fig. 515.—Tensile Strength of Rosendale (Natural) Cement Mortar. (Baker, in *Masonry Construction* p. 90.)

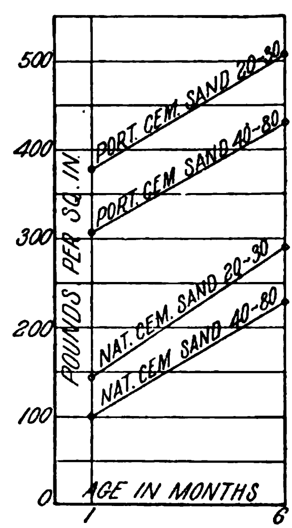

Fig. 516.—Effect of Size of Sand-grains on the Strength of Cement-mortar, 3 S.: 1 C. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 3013)

M. Feret* has for many years made a study of the effects of the "granulometric" composition of the sand on the various qualities of the

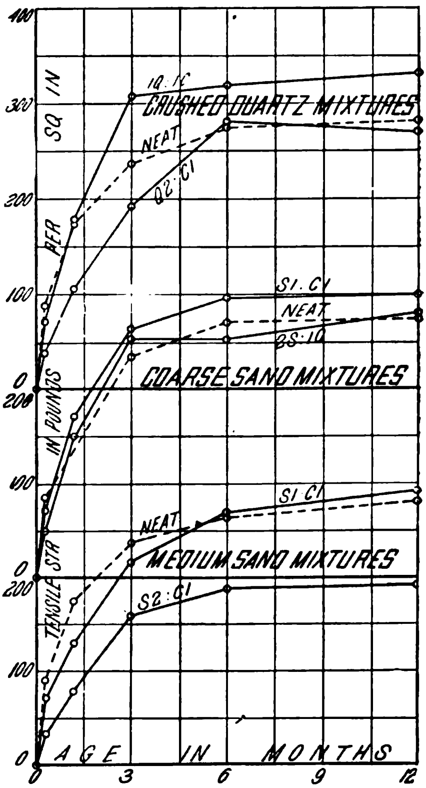

FIG. 517.—Time Tests on Three Kinds of Portland Mortar with Different Sands. (*R. R. Gazette*, 1892.)

resulting mortars.† He experimented with two sands of the following granulometric compositions:

Kind of Sand.	Large Grains, 2.0 mm. to 5.0 mm.	Medium Grains, 0.5 mm. to 2.0 mm.	Fine Grains, passing 0.5 mm.
Coarse (Gattemarre).................	52%	48%	0
Fine (Trouville)....................	1%	24%	75%

The granulometric composition was found by sifting through thin plates having circular holes of 5 mm., 2 mm., and 0.5 mm. respectively, with the

* Chef du laboratoire des Ponts et Chaussées à Boulogne-sur-Mer, France.

† See his papers in *An. d. Ponts et Chaussées* for Mar. 1890, July 1892, Aug. 1896, and in *Baumaterialenkunde*, vol. I, No. 10.

FIG. 518.—Granite, size 80–100.

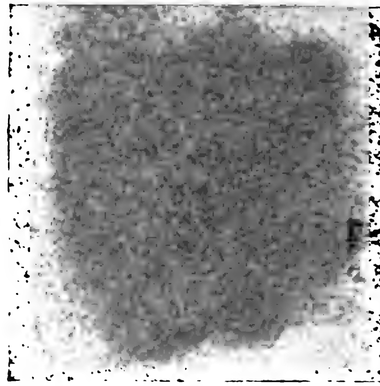

FIG. 519.—River-sand, size 120-140.

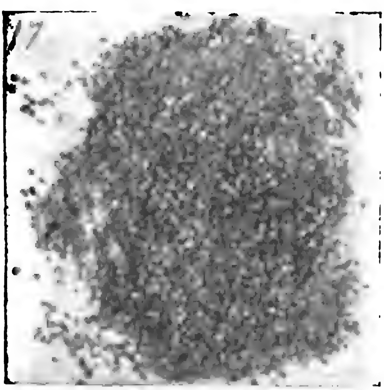

FIG. 520.—River-sand, size 20–30.

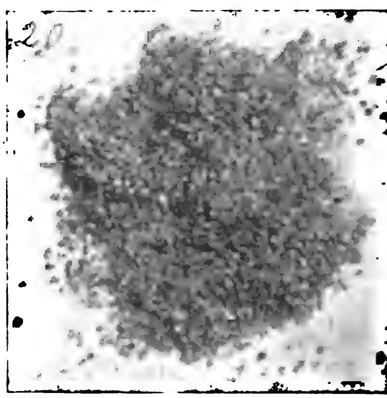

FIG. 521.—River-sand, size 20-30.

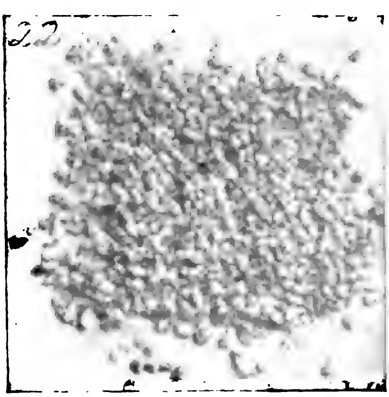

FIG. 522.—Granite, size 12-16.

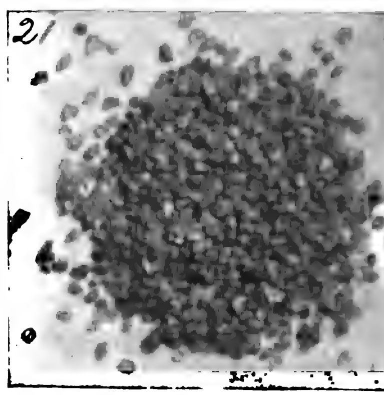

FIG. 523.—River-sand, size 12–16.

Photographs of Sand-grains. Natural Size. (Cooper, in *Jour. Frank. Inst.*, vol. CXL. 1896.)

results as indicated by the table above. When all possible proportions of coarse and fine sand had been tried with a cement ingredient varying from 10 to 30 per cent of the total, it was found that *the strongest mortar for any given percentage of cement was always found for a weight of coarse sand equal to twice the combined weight of the fine sand and the cement.* With this condition fixed, the strength and cost of all mortar mixtures fulfilling this condition are given in Fig. 524.

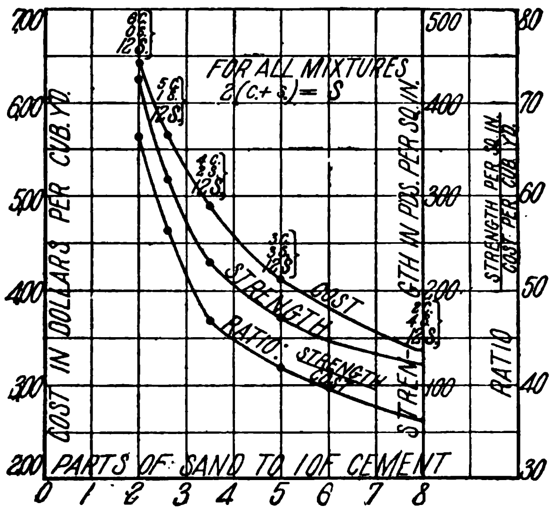

Fig. 524.—M. Feret's Maximum Strength Mixtures in which the Coarse Sand (S.) is twice the Combined Weight of the Fine Sand (s.) and the Cement (c.). (*An. d. Ponts et Chaussées*, Aug. 1896, p. 191.)

1-3

411. Relative Economy of Coarse and Fine Sand in Cement-mortars.— When the choice lies between a coarse sand and a fine sand exclusively for use in cement-mortar for any purpose, the preference should always be given to the coarse sand, even though its cost is many times that of the fine sand. Thus M. Feret gives as a generalization from his years of experimentation on this subject* a table from which Fig. 525 has been constructed. Here we have as a common argument the compressive strength of the mortar mixtures, at the age of three months, for any given brand of Portland cement. In this figure we have for the two sands whose granulometric composition is given in the note below the figure:

1. Weight of cement to use with one cubic yard of coarse sand to produce a mortar of any given strength.
2. The same for fine sand.
3. Weight of cement to use to produce one cubic yard of mortar of any given strength when coarse sand is used.

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* In *Les Matériaux de Constructions* (*Baumaterialenkunde*), vol. I. p. 189.

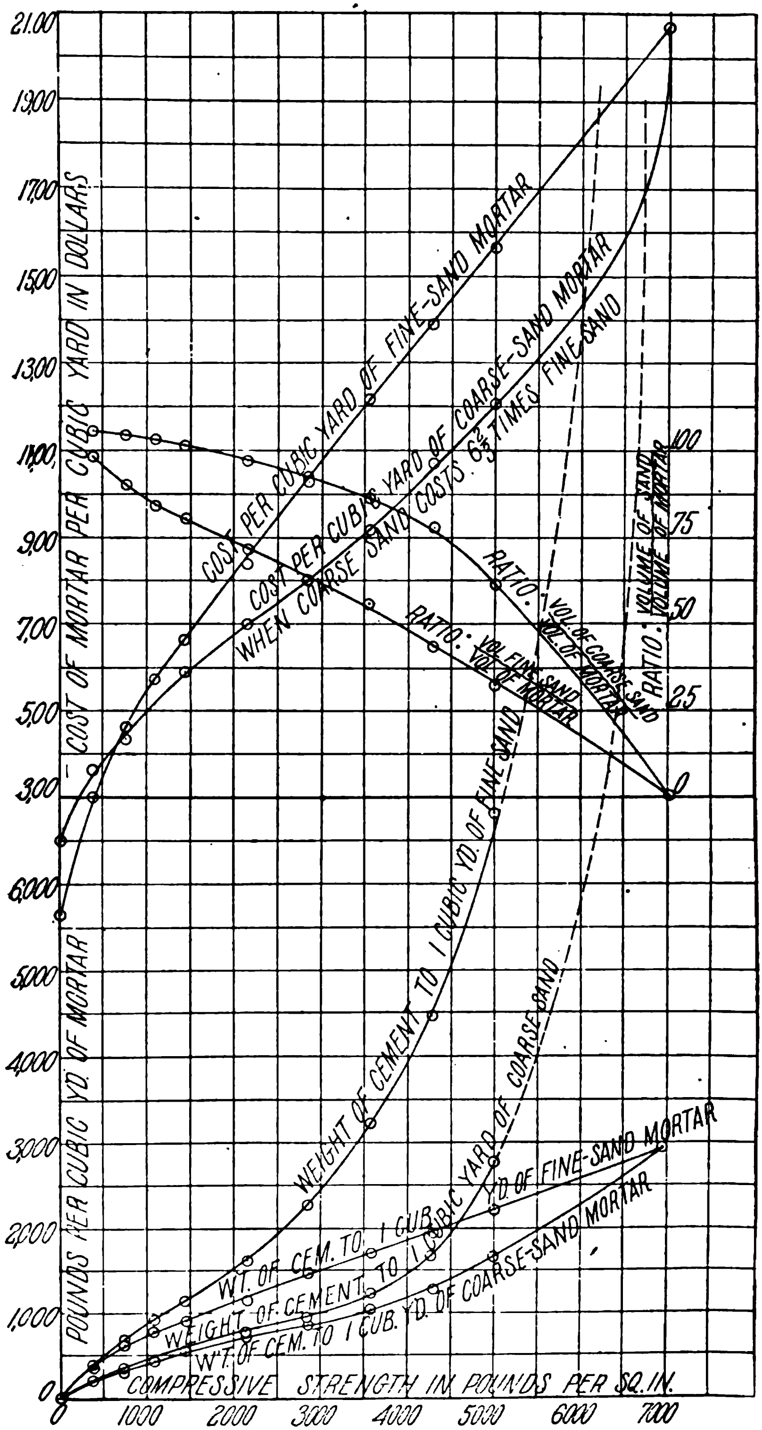


Fig. 525.—Relative Economy of Coarse and Fine Sand in Portland-cement Mortar 1 C. : 3 S. after Five Months' Immersion in Sea-water. Coarse sand composed of 52% 2 mm. to 5 mm. diam.; 48% 0.5 mm. to 2 mm. diam. Fine sand, 25% 0.5 mm. to 2 mm.; 75% less than 0.5 mm. diam. (M. Feret, in *Les Matériaux de Construction,*

4. The same when fine sand is used.
5. Cost per cubic yard of coarse-sand mortars of any given strength.
6. The same for fine-sand mortars *when the fine sand costs only fifteen per cent as much as the coarse sand.*
7. The ratio of the volume of coarse sand to the volume of the mortar of any given strength.
8. The same when fine sand is used.

From these diagrams the following remarkable conclusions may be drawn:

A. *It requires about twice as much cement mixed with a given quantity of sand to produce a mortar of given strength when fine sand is used as it does with coarse sand*

B. *The weight of cement per cubic yard of mortar of a given strength is about twice as much for fine sand as for coarse sand, with the ordinary mixtures.*

C. *The cost per cubic yard of coarse-sand mortar of a given strength (such as is found for the ordinary ratio* 1 C. : 3 S.) *is only about seventy-five per cent of the cost of a fine-sand mortar of the same strength, even when the coarse sand costs six and two-thirds times as much as the fine sand (coarse sand* \$1.30, *and the fine sand* \$0.20 *per cubic yard*).

412. Experiments with Sands of Artificial Granulometric Composition.—Very coarse or gravelly sands, containing pebbles as large as one-fourth inch in greatest dimension, may be introduced into a mortar used in making concrete, or in rough masonry, with great economic advantage. M. Feret has studied the effects of the use of such sands, mixed in various proportions with finer grades, and some of his results are given in Figs. 526 to 531 He used for these experiments three grades of sand, namely:

Grade of Sand.	Passes a Perforated Plate having Holes of a Diameter of	Is Stopped on a Plate having Holes of a Diameter of
Coarse sand....................	5 mm. or 0.2 in.	2 mm. or 0.08 in.
Medium sand....................	2 mm. or 0.08 in.	0.5 mm. or 0.02 in.
Fine sand......................	0.5 mm. or 0.02 in	....

He made all possible mixtures of these three grades, representing each mixture by its position in an equilateral triangle, just as has been done in the case of the bronzes in Fig. 76. Thus in Fig. 526 let each apex of the triangle represent 100 per cent of one kind of sand, and on perpendiculars drawn from these points to the opposite sides let percentages be marked, reducing to zero on those sides as shown in the figure, and let lines be drawn through these points parallel to the several sides as shown. Then may any particular composition of sand, made up of any given proportions of the three grades, be represented by the position of a point which shall be distant from the several sides by amounts equal to the three percentages, as indicated on the normals to these sides. This follows from the geometrical

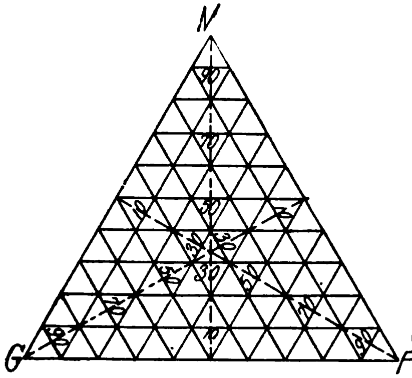


FIG. 526.—Showing the Method of Representing Proportionate Mixtures of Three Ingredients. G = coarse sand, 0.2 in. to 0.08 in. in diameter. M = medium sand, 0.08 in. to 0.02 in. in diameter. F = fine sand less than 0.02 in. in diameter.

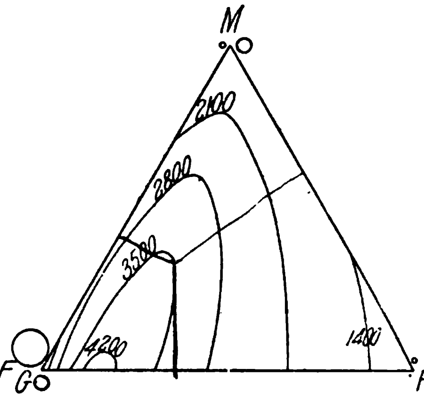


FIG. 527.—Compressive Resistance of Portland-cement Mortars, in pounds per square inch, after nine months in air and then three months in sea-water. Mortar 1 C. : 3 S. in all cases, but the composition of the sand varying according to position in the triangle.

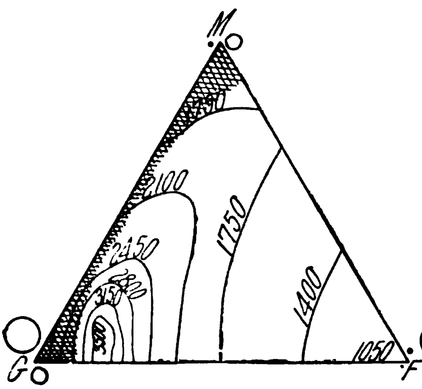


FIG. 528.—Compressive Resistance of Portland-cement Mortars, 1 C. : 3 S., in pounds per square inch, *after one year in sea-water*. Shaded part indicates mixtures which were partially disintegrated.

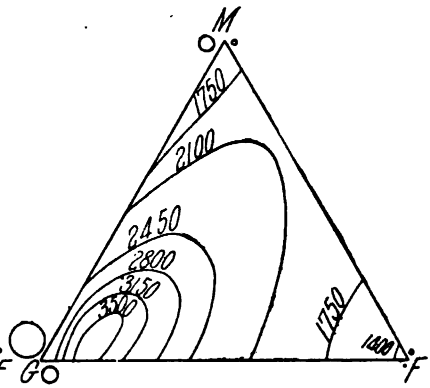


FIG. 529.—Compressive Resistance of Portland cement Mortars, 1 C. : 3 S., in pounds per square inch, *after one year in fresh water*.

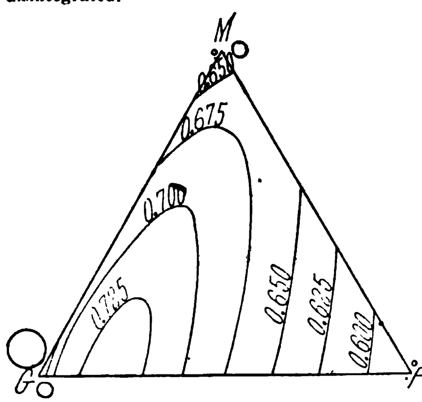


FIG. 530.—Actual Solid Contents (C. + S.) of Portland-cement Mortars, 1 C. : 3 S., in terms of the total bulk of the mortar.

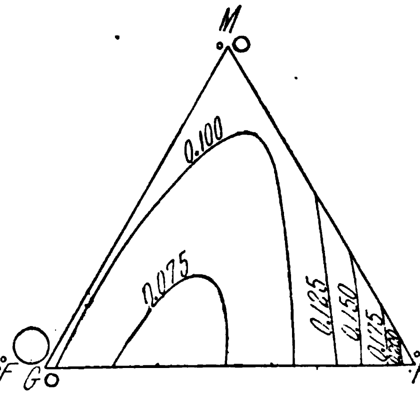


FIG. 531. — The Porosity of Portland-cement Mortars, 1 C. : 3 S., as indicated by the percentage of water absorbed by the mortar after it had hardened and dried.

Samples of M. Feret's Diagrams illustrating Effects of Varying the Granulometric Composition of the Sand used in making Portland-cement Mortars. The actual sizes of the Sand-grains of the Three Ingredients are indicated by the small circles at the corners of the triangles. (Ann. d. Ponts et Chaussées, series 7, vol. ..., (1892)

proposition that the sum of the three normals from any point in an equilateral triangle to the three sides is equal to the common altitude of the triangle. The various characteristics of such mortars may now be represented by lines drawn upon these triangles, which shall join points of equal numerical value in the quality under consideration, the same as contour-lines on a map join points of equal elevation above a given datum plane.

Thus in Fig. 527 the compressive resistance in pounds per square inch is indicated for all possible mixtures of these three grades of sand, there being in all cases a total of 3 S. to 1 C. by weight. These samples were all left nine months in the air and then three months in sea-water. From this figure we conclude:

A. That a sand composed of 4 parts of very coarse sand (0.08–0.20 in. diam.) to 1 part of very fine sand (less than 0.02 in. diam.) makes the strongest possible mortar of 1 C. : 3 S.

B. That the strength of such a mortar is more than twice as much as the same mortar 1 C. : 3 S. when the sand is composed of what is commonly regarded as "coarse sand" (0.02–0.08 in. diam.), and more than three times as strong as the same (1 C. : 3 S.) mortar when the sand is very fine (less than 0.02 in. diam.).

C. That a mixture of two grades of sand of widely different sizes gives a great deal stronger mortar for given proportions of sand and cement than does any particular size when used by itself.

D. It follows from the above that it is well to employ as coarse a sand as the work will admit of, even to the finer gravels in the case of coarse masonry, and especially with the concretes.

E. It follows also that in the case of concrete mixtures it is well to leave in the smaller sizes of the crushed rock, provided the very fine particles be excluded. This has been found to be the case in actual practice.

F. That it would pay to use very coarse sand at a very much higher price than to use medium or fine sand at a low price, or even if its cost be *nil*.

Very similar results to the above are shown in Figs. 528 and 529, from which like conclusions may be drawn. The shaded part of Fig. 528 indicates that for these mixtures, after exposure to sea-water for one year, there were some signs of disintegration, due doubtless to the greater permeability of these mixtures. (A distinction must be drawn between *permeability* and *porosity*. See next article.)

413. The Porosity of Mortars as affected by the Size of the Sand-grains.—Figs. 530 and 531 indicate the relative and absolute porosity of various sand mixtures as affected by the granulometric composition of the sand used. Thus Fig. 530 gives the actual solid contents, per unit volume of mortar, of the cement and sand combined which entered into the composition. Fig. 531 gives the volume of water absorbed, per unit volume of the dry mortar, for all granulometric compositions of the sand. In both cases the greatest porosity is found with the finer grades of sand, and the least for a mixture of two of very coarse (gravelly) sand to one of fine sand.

The relative permeability cannot be assumed to vary with the porosity, since a given degree of porosity with coarse sand produces a much more permeable mortar than the same degree of porosity with fine sand. Hence

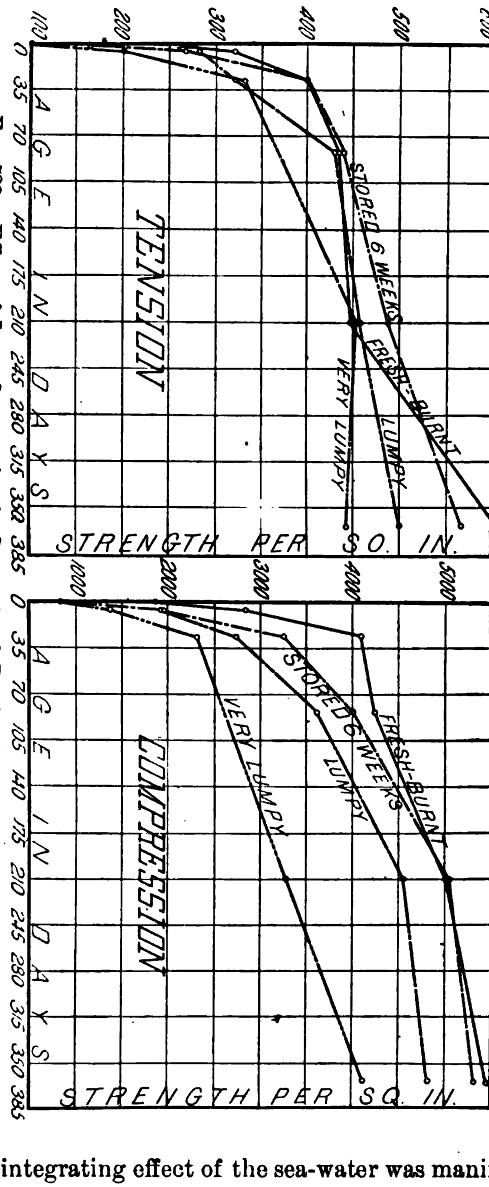


Fig. 532.—Effect of Long Storage in the Strength of Portland-cement Mortar, 1 C. : 3 S. (Tetmajer, vol. VII. p. 15.)

in Fig. 528 the disintegrating effect of the sea-water was manifested with the coarse-sand mixtures, while the fine-sand mixtures did not reveal any such action, although its porosity was much greater.

414. The Effect of Long Storage on the Strength of Cement.—The effect of long storage is to reduce the strength of the cement more or less, whether this be Portland, natural, or slag cement. The injury, however, is not as great as is commonly supposed. Thus in Fig. 532 we have both tensile and compressive tests on standard Portland-cement mortar, 1 C. : 3 S., for various conditions from "fresh burnt" to "very lumpy." The loss of tensile strength seems to be less than the loss of compressive strength, though, except in the latter case for the "very lumpy," the loss is not material.

In the case of natural cement over thirty days old the loss of tensile strength is considerable, as shown in Fig. 533. Here, however, the cement had been spread out and exposed to the air.

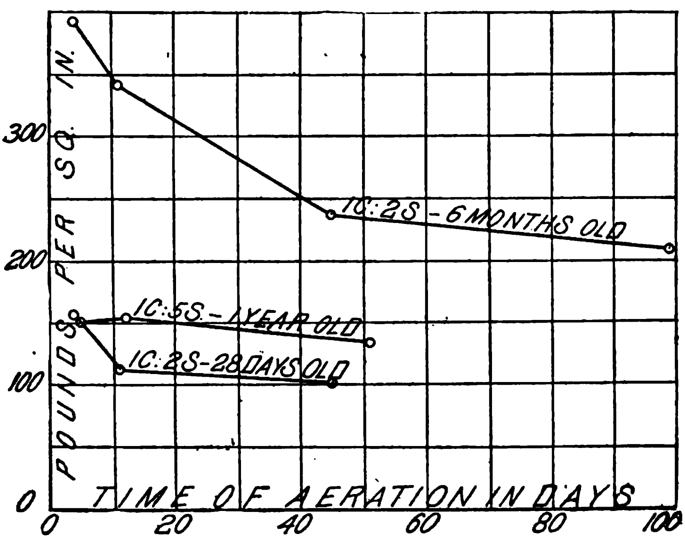
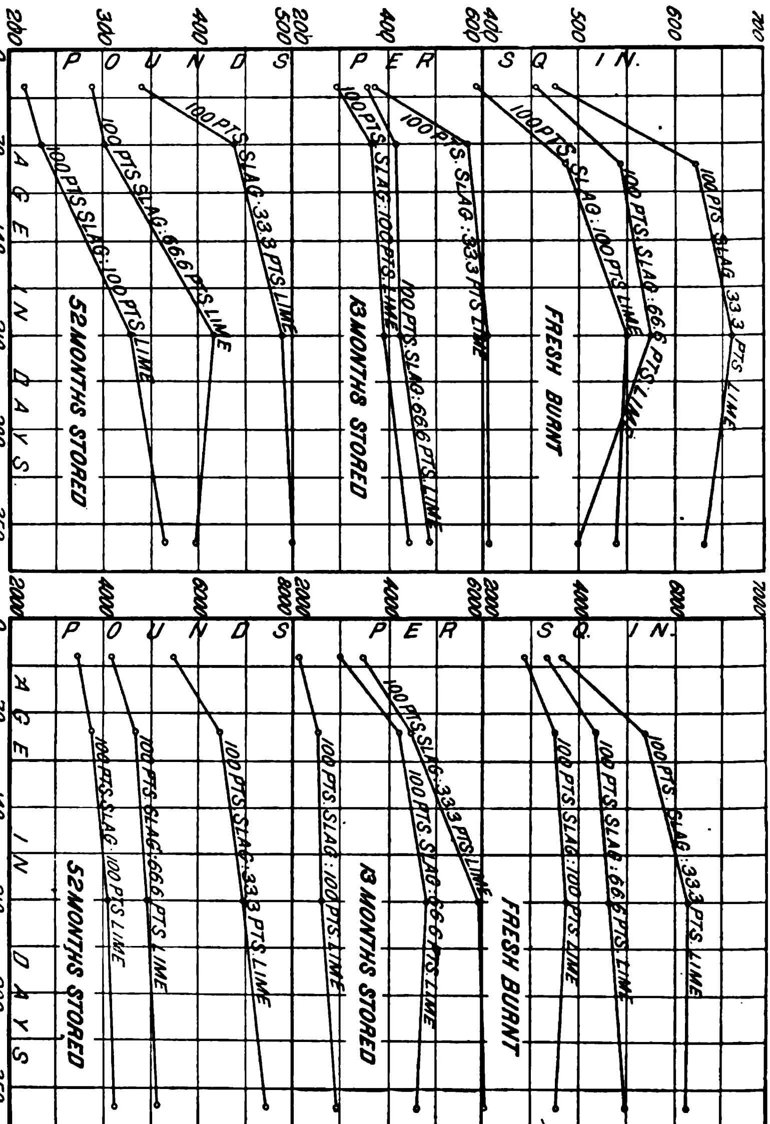


FIG. 533.—Effect of Aeration on the Strength of Natural-cement Mortar. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2962.)

In both the natural and in the slag cements there is free lime, which changes to the inert carbonate of lime when exposed to air containing carbon-dioxide gas. The result is to destroy this lime ingredient. In other cases, where there is unslacked free lime, a long aeration allows this ingredient to slack, and so prevents this action in the hardened cement, which would swell and crack it. It is the object of the boiling test to detect the presence of any such slow-slacking free lime in the cement. The effects of long storage of slag-cement containing various proportions of free lime on both the tensile and the compressive strength are shown in Fig. 534.

415. Effect of Regauging after Set Begins.—It is commonly understood that cement which has begun to set is more or less weakened by regauging,

and that such cement should never be used in practice. The results shown in Figs. 535, 536, and 537 reveal to what extent the mortar is weakened. Thus from Fig. 535 it may be seen that a quick-setting natural cement



TENSION.

COMPRESSION.

FIG. 534.—Effect of Storage on Slag-cement containing Lime in a Moist Air containing Carbonic-acid Gas. (Tetmajer, vol. VII. p. 116.)

mixed neat loses over 25 per cent of its strength at six months from having been regauged once one hour after wetting. When regauged repeatedly in 3 or 5 hours it loses 40 per cent of its normal strength.

From Fig. 536 it appears that a Louisville-cement mortar 1 C. : 2 S. loses 40 per cent of its normal strength at three months by standing 20 minutes after wetting before moulding, and 80 per cent of its normal

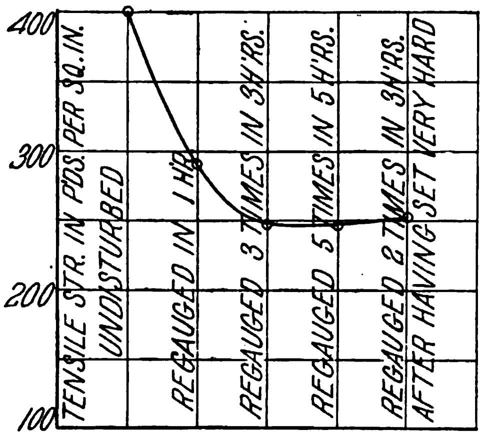


FIG. 535.—Effect of Regauging a Quick-setting, Neat Natural-cement Mortar, Age Six Months. (*Rep. Chf. Engrs.* 1895, p. 2980.)

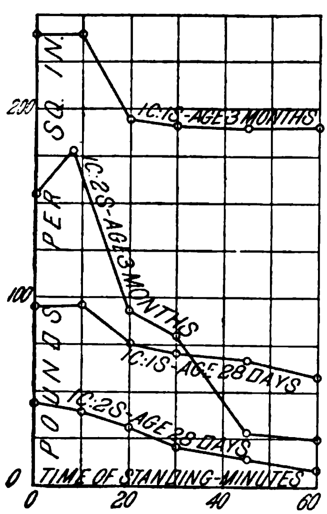


FIG. 536.—Effect on the Strength of Louisville-cement Mortar of allowing it to stand a given time before putting into the moulds. (*Jour. West. Soc. Engrs.*, vol. I. p. 82.)

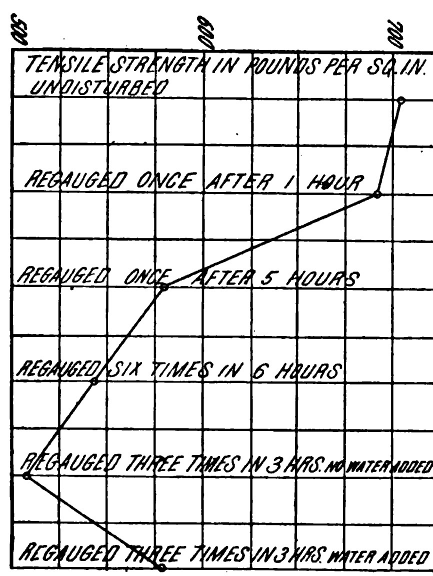
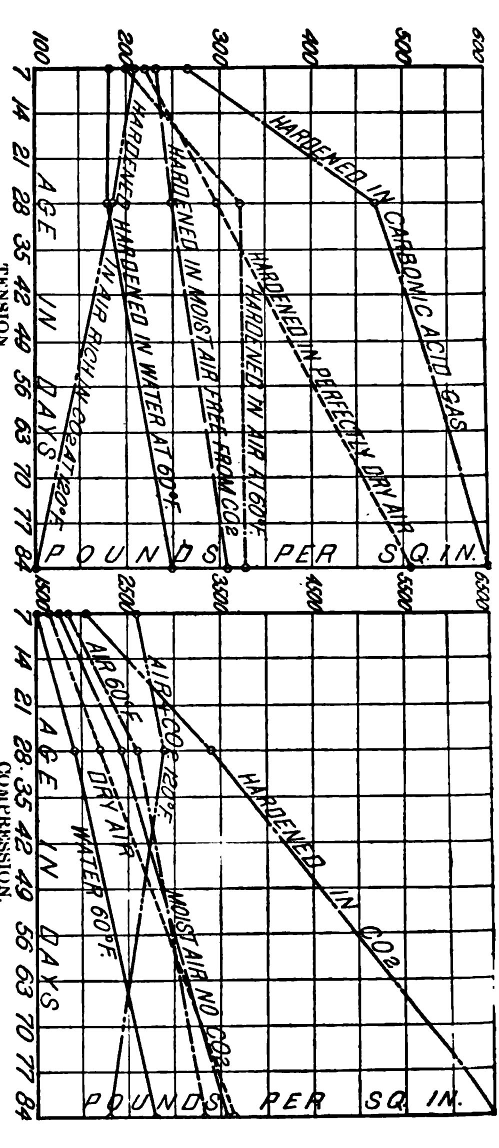


FIG. 537.—Strength of Regauged Neat Portland Cement after Six Months' Hardening in Water. Time of setting: begins in 50 min., ends in 3 hrs. 25 min. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2979.)

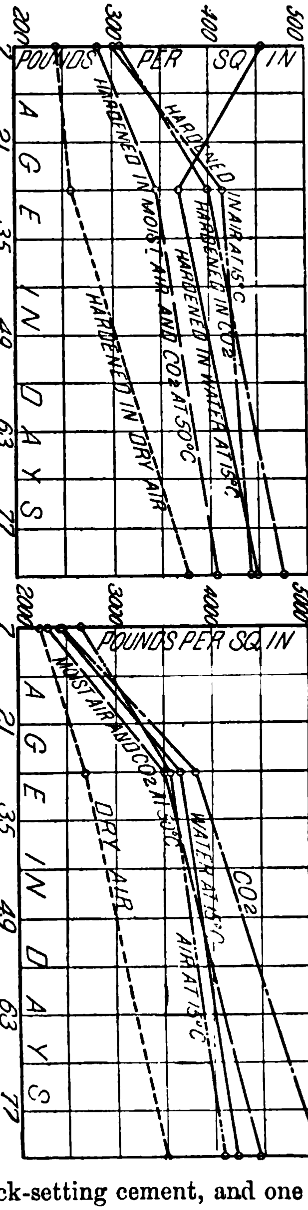
strength by standing one hour before moulding. The loss of strength in the 1 C. : 1 S. mortar, though serious, is not so great. This is, however, a very



TENSION.

COMPRESSION.

FIG. 538.—Effect of Carbonic-acid Gas on the Hardening of Natural-cement Mortar, 1 C. : 3 S. (Tetmajer, vol. VII.)



TENSION.

COMPRESSION.

FIG. 539.—Effect of Carbonic-acid Gas on the Hardening of Slag-cement Mortar, 1 C. : 3 S. (Tetmajer, vol. VII. p. 20.

quick-setting cement, and one which should evidently be used inside of 20 minutes from the instant of wetting it.

The loss of strength from regauging one or more times a neat Portland-cement mortar, during a period of from one to six hours, is shown in Fig.

537. This cement completes its set in 3 hours 25 minutes, and loses 18 per cent from being regauged once in five hours or three times in three hours. Evidently any cement will be greatly weakened if used after it has set.

416. Effect of Carbonic-acid Gas on the Hardening of Natural- and Slag-Cement Mortars.—These two classes of cement have more or less free lime in their composition, and the action of the CO_2 on this is to change it into the carbonate, $CaCO_3$ (limestone), which would naturally add to the strength of the mortar. As Portland cement does not contain free lime to any appreciable extent, it would not be similarly affected. In Fig. 538 the effect of CO_2 on a natural-cement mortar, 1 C. : 3 S., is shown to be very great on both the tensile and the compressive strength.

From Fig. 539 the effect on slag-cement is not so great, although quite marked. It may further be observed from this dragram that while hardening in perfectly dry air is very favorable to the natural cement, it is very unfavorable to the strength of the slag-cement. This would also be found to be the case with Portland-cement mortar. Why hardening in moist air at 120° F. (50° C.), which is rich in CO_2, should be so very unfavorable to the strength of natural cement does not appear.

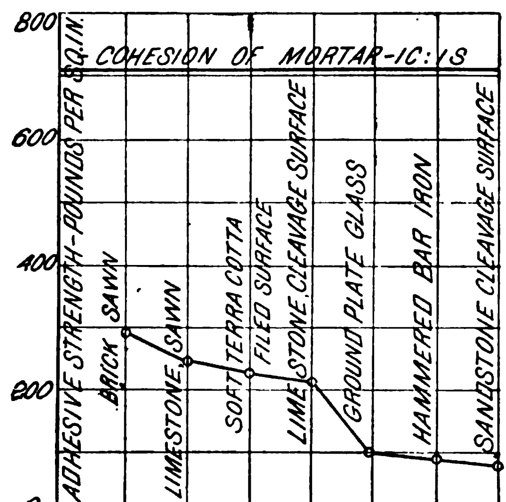


FIG. 540.—Adhesive Strength of Portland-cement Mortar, 1 C. : 1 S., Twenty-eight Days Old, to Different Substances, and the Cohesive Strength of the Mortar itself. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 3019.)

417. The Adhesive Strength of Cement-mortars.—This is a subject of very great importance, but one which has not been commonly investigated. It is to be hoped that the standard methods proposed for this test by the French Commission will lead to further experiments giving comparable results.

In Fig. 540 the adhesive strength of Portland-cement mortar, 1 C. : 1 S., is given for various substances. Here small disks of the substance, 1 inch square and ¼ inch thick, were prepared and inserted transversely at the centre of the briquette-mould, and the briquette pulled in the usual manner, with the results as shown. It thus appears that, whereas the cohesive strength of this mortar was 710 lbs., its adhesive strength varied from 300 lbs. on sawn brick to 85 lbs. per square inch on sandstone having a cleavage surface.

In Fig. 541 it is shown that while Portland-cement mortar will adhere

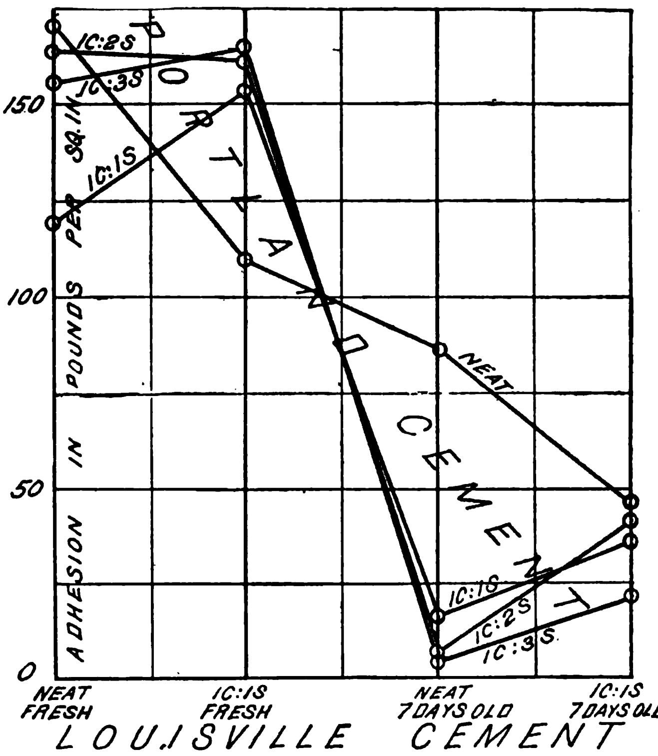
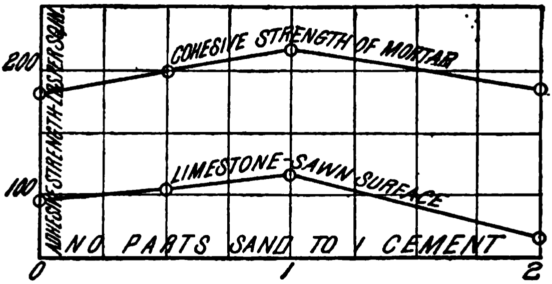


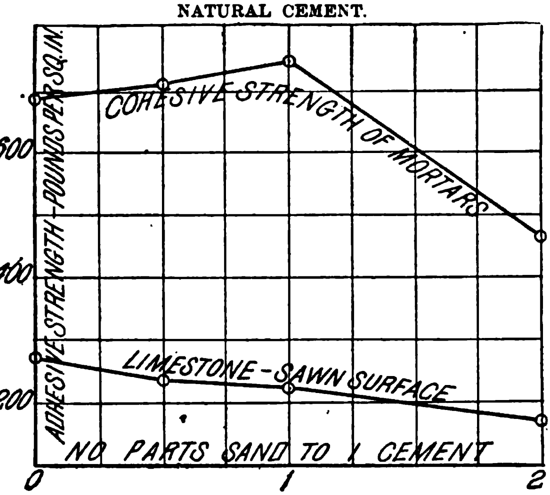
FIG. 541.—Adhesion between Louisville (Natural) and Portland Cement Mortars. (*Jour. West. Soc. Engrs.*, vol. I. p. 82.)

to natural (Louisville) cement mortar when both are fresh, it will scarcely adhere at all to a neat natural-cement surface after it is seven days old, and it adheres very poorly to a 1 C. : 1 S. natural-cement mortar a week old. The neat Portland cement did adhere to the neat Louisville cement one week old with a force of 85 lbs. per square inch, but the Portland-cement sand-mixtures would not adhere to it with any appreciable force.

The adhesion of natural and Portland cement mortars to sawn limestone, as compared with their cohesive strength, is shown by the diagrams in Fig



NATURAL CEMENT.



PORTLAND CEMENT.

FIG. 542.—Relation between the Adhesive Strength at Twenty-eight Days of Cement-mortars to Sawn Limestone and the Cohesive Strength of the Mortars themselves. (Wheeler, *Rep. Chf. Engrs.* 1895. pp. 3020–21.)

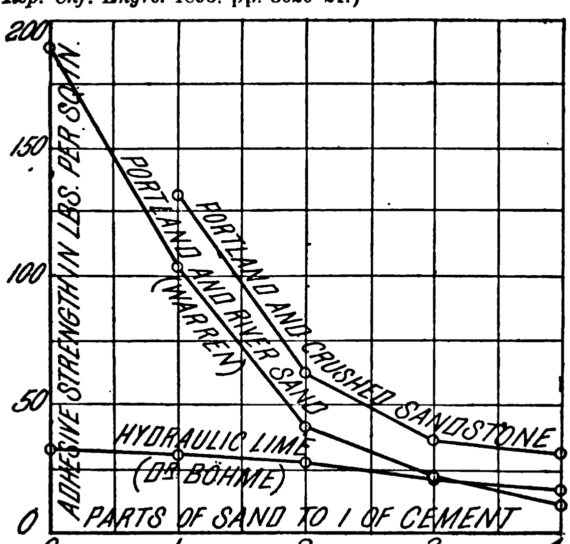
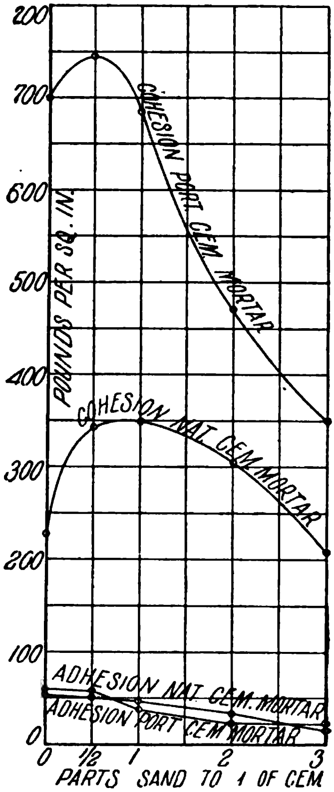


FIG. 543.—Adhesive Strength of Mortar to Brick Surfaces. (*Baker's Masonry*, p. 94.)



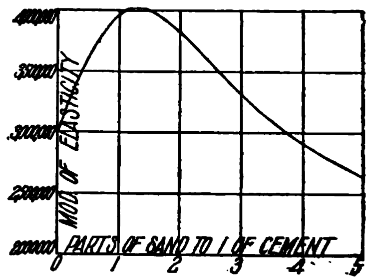


Fig. 544a.—Showing the Variation in the Modulus of Elasticity of Portland-cement Mortar in Compression, at the age of Three Months when tested in Cylinders 10 in. in Diameter and 40 in. long. (Prof. C. Bach in *Zeits Ver. Deuts. Ing.*, Nov. 28, 1896.

Fig. 544.—Relation between the Cohesive Strength of Cement-mortar and its Adhesive Strength to Brick Surfaces when Two Bricks are cemented together in cruciform shape and pulled normally. The results are the means of three months' and six months' tests on both die and stock brick. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 3022–4.)

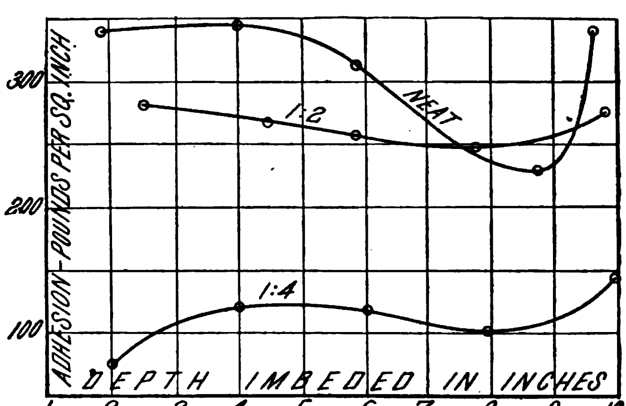


Fig. 545.—Adhesion of Plain 1-inch Round Bolts in Neat Portland-cement Mortar, Age One Month. Adhesion given in pounds per square inch of surface of bolt embedded. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2941.)

542. In general we may say the adhesive strength here shown is less than one half the cohesive strength.

The adhesive force of ordinary cement-mortars to brick surfaces is very small, as shown by Figs. 543 and 544. A strength of 25 lbs. per square inch seems to be about all that can be ordinarily counted on This low adhesive strength may be partly due to the fact that the bricks are covered with a coating of disturbed and loose particles. A clean, fresh fracture would probably show a much greater cohesion.

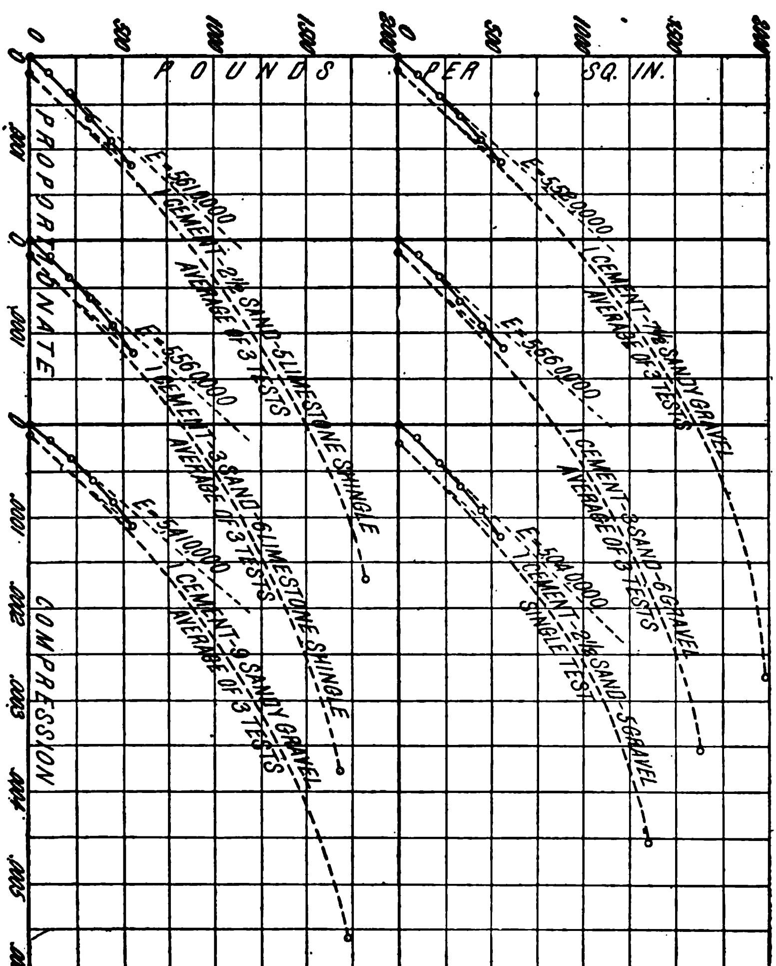


FIG. 546.—Compressive Strength of Portland-cement Concrete Blocks Ten Inches in Diameter and Forty Inches Long, Three Months Old. The deformations were not observed on the broken lines. (Prof. Bach in *Zeits. Ver. Deutscher Ing.*, April 27, 1895.)

The adhesion of cement-mortar to anchor-bolts embedded in stone is very great, as shown by Fig. 545. These tests agree well with experiments made by the author. The ultimate strength at three or six months would be

nearly twice as much as shown in the diagrams, which are for an age of four weeks. The same department obtained about twice the adhesive strength shown in Fig. 545 by using limestone screenings, passing a ⅜-inch-mesh sieve,

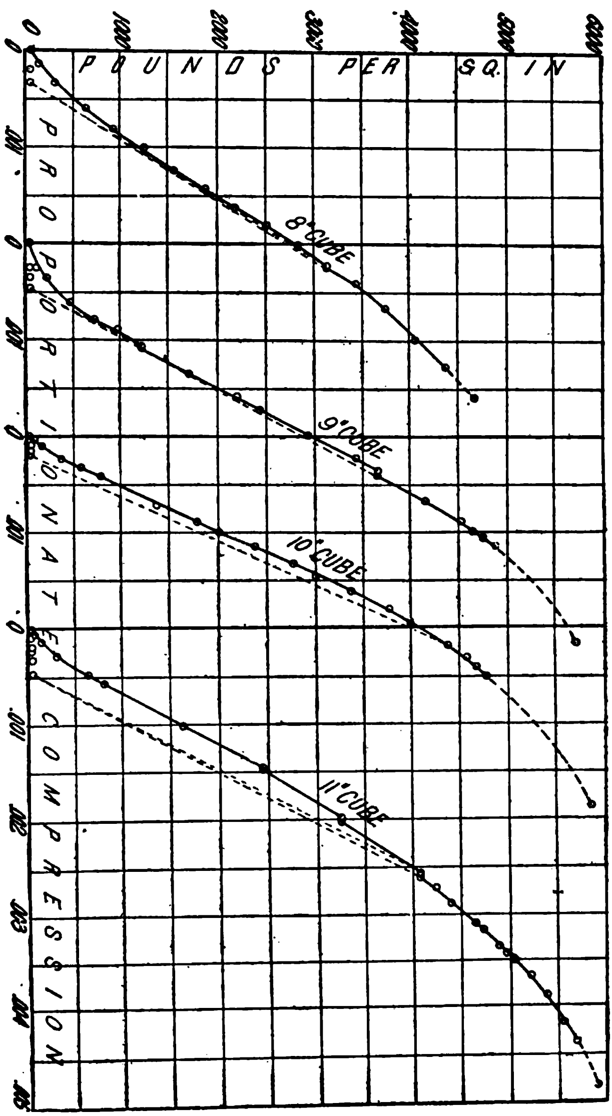


Fig. 547.—Compression Tests on Cubes of Dykerhoff Neat Cement Two Years Old. (Gilmore.)

using 2 S. to 1 C. Hence the ultimate adhesive strength of a good Portland cement with limestone screenings, 1 C. : 2 S., to plain iron or steel bolts, may be taken at about 1000 lbs. per square inch. To develop a working strength on anchor-bolts, therefore, of 20,000 lbs. per square inch would re-

quire a depth of adhesive surface equal to 20 diameters of the bolt. To provide a sufficient factor of safety a depth of 30 or 40 diameters should be used.

418. Compressive Strength and Elasticity of Cement and Concrete.—In Fig. 546 are shown the results of Prof. Bach's tests on concrete columns 10

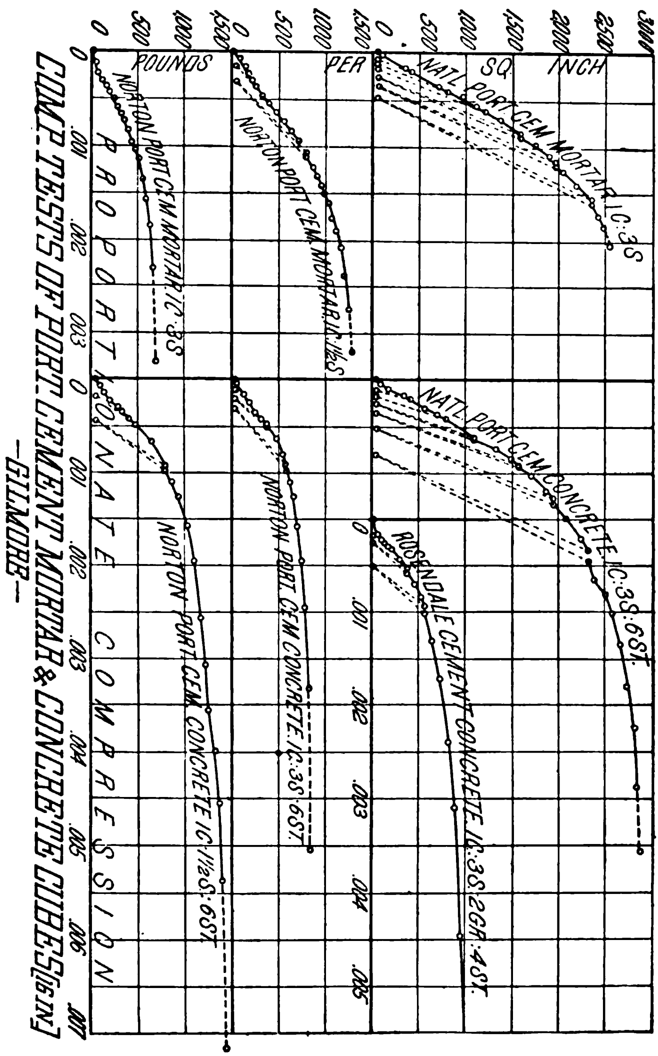


FIG. 548.—Compression Tests on Cement-mortar, 16-in. Cubes. The Portland-cement cubes were four years old, and the Rosendale-cement cubes were two years old, when broken. (Gilmore.)

inches in diameter and 40 inches long. The deformations of these blocks were obtained with the apparatus shown in Fig. 261, p. 356. These blocks were composed of Portland cement, sand, and gravel, and they exhibit two remarkable characteristics. They do not give the reverse-curved stress-

diagrams commonly obtained with concrete and stone blocks and shown in Figs. 547 and 548, and they also give very high moduli of elasticity. The

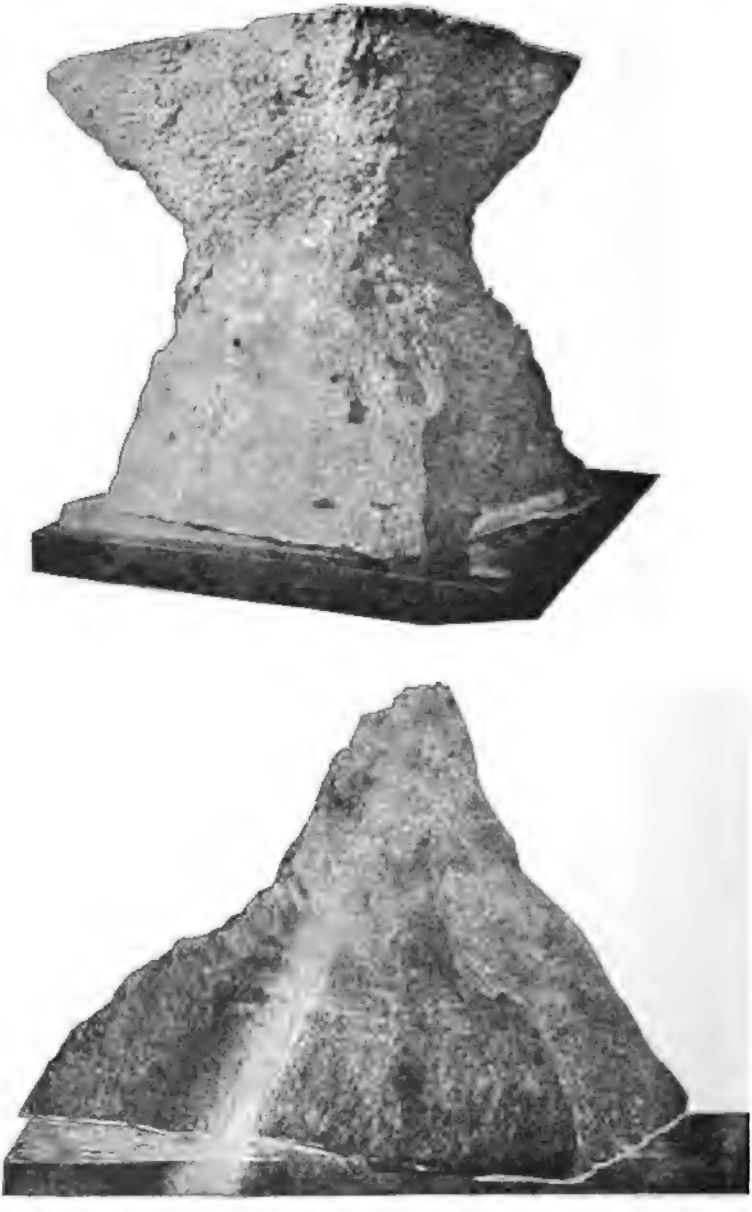


FIG. 549.—Showing Method of Failure of Cement Cubes. (*Wat. Ars. Rep.* 1884.)

author has not found any other tests giving these two characteristics in so marked a degree. The mixtures were doubtless very carefully made.

Figs. 547 and 548 are compiled from Gilmore's *Limes, Mortars, and Cements.* Here the modulus of elasticity is only from one third to one half that obtained by Bach,* and there is always a large permanent set under the earliest, or smallest, loads. Probably this is due to imperfect mixing or to poor compacting in the moulds, or to both. Gilmore's tests were made on the Emery machine at the Watertown Arsenal. The manner in which these cubes fail under compression is shown in Fig. 549. Bach's moduli of elasticity on cement-mortar columns are given in Fig. 544*a*, p. 600.

419. Strength and Economy of Cement-mortar and Concrete.—There is no very uniform practice in America in the number of parts of sand to one of cement to be used in mortars. In general natural cement is used with one or two parts of sand, while Portland cement is commonly used with three parts of sand to one of the cement by measure. Whether the cement is to be measured in the original packages or in the loose condition it assumes when turned out is a matter of great significance, but no uniform practice is followed, and usually the specification is defective in not defining which method is to be employed. The amount of sand which may be used with a given cement depends on the percentage of voids in the sand, and on the fineness of the cement. If the sand-grains are graded in size, the voids are a smaller proportion; and if the cement is all finely ground, it is all active. Such parts of the cement as will not pass a No. 120 sieve (14,400 meshes per lineal inch) has no value as a cement and acts as so much sand.

In Fig. 550 are shown the results of an excellent series of tests made by Mr. E. S. Wheeler, M. Am. Soc. C. E., in connection with the building of the St. Mary's Falls Canal lock. Here the natural-cement mortars all give the greatest strength for a given cost, the price of the natural cement being 43 per cent of that of the Portland cement, delivered on the works. The most economical natural-cement mortar is that of 1 C. : 2 S. or 1 C. : 3 S. Probably 1 C. : 2½ S. is the best mixture for natural cement. With the Portland cement the mixtures 1 C. : 2 S., 1 C. : 3 S., and 1 C. : 4 S. were all about equally strong for a given cost, or, what is the same thing, these mixtures are about equally expensive for a given strength.

Evidently an ideal concrete is one in which all voids are filled, all sand-grains are coated with cement, and all pebbles, gravel, or broken stones are coated with mortar, with no excess of cement or mortar. This requires that enough cement must be used to fill the voids in the sand (plus some excess to cover imperfect mixing), and enough mortar used to fill the voids in the stone or gravel (plus an excess as before). In the Report of the Chief of Engineers of the U. S. Army for 1895,† pp. 2924 to 2931, will be found

* To read the modulus of elasticity from any of the curves in Figs. 546, 547, or 548, find the change of load per square inch for which the proportionate compression is 0.001, and multiply such change in load by 1000, using the straight portion of the diagrams for such readings.

† Under the direction of Mr. E. S. Wheeler, U. S. Ass't Eng'r in charge of the construction of locks on the St. Mary's Falls Canal.

the records of the most complete series of tests of the cross-breaking strength of concrete beams ever made. These beams were all 10 inches square, and were broken on a span of 4 feet. There are here recorded the results of tests on over one hundred such beams, forty of which were again broken on a span of 20 inches. The first breaks were at one year old, and the second at 22 months. All kinds of mixtures and conditions were used in the making of the beams, and they were covered by moist earth during the entire hardening period, or until broken.* In all these tests the proportions by both

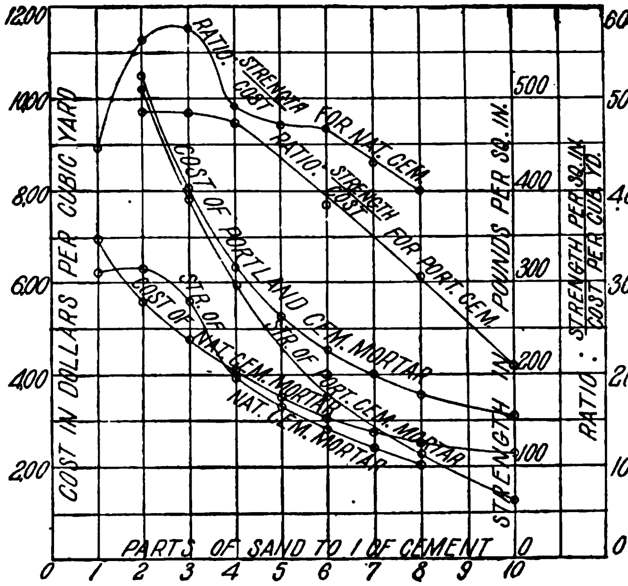


FIG. 550.—Relation between Strength and Cost of Natural- and Portland-cement Mortars. Prices: Nat. cem. = \$1.30 per bbl.; Port. cem. = \$3.00 per bbl.; sand = \$1.00 per cu. yd. (Wheeler, *Rep. Chf. Engrs.* 1893, vol. IV. p. 3022.)

weight and by volume are recorded, and the cost of each part, and many other pertinent facts. A few of these results, which were of the nature of a series, are plotted in Figures 551 to 554, and the full record of the tests is given in Table XXXVII.

In Fig. 551 the cost per cubic yard and the cross-breaking strength in pounds per square inch are given for various mixtures of Portland-cement concrete. From this diagram the most economical mixture does not appear. Evidently the greatest economy corresponds to the greatest ratio of strength to cost, or, what is the same thing, the minimum ratio of cost to strength. This may be shown by plotting one of these ratios to the number of parts of sand and stone to one of cement. This is done in Fig. 552. From this it appears that the most economical mixture of Portland-cement concrete (the

* The series was not completed at the time of the 1895 report, and the 1896 report will contain further results.

prices being \$3.00 per barrel for cement, 50 cents per cubic yard of sand, and \$1.00 per cubic yard of stone or gravel) is 1 C. : 3 S. : 7½ broken stone or gravel, all by volume.* The cross-breaking modulus of rupture of this

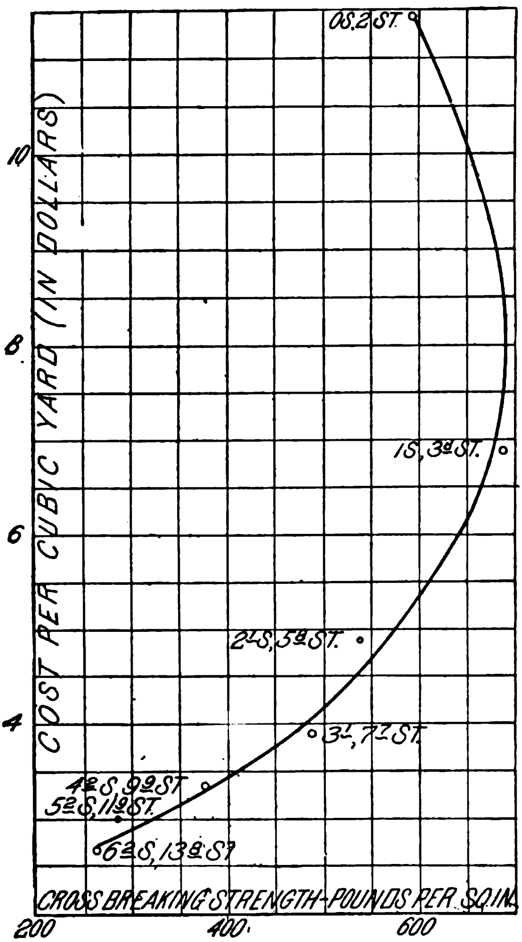


FIG. 551.—Relation between Cost and Strength of Portland-concrete Beams Nineteen Months Old. Each result is the mean of two tests on beams 10 in. square. Portions of sand and stone are given to 1 of cement. Cement \$3 00 per bbl.; sand \$0.50 and stone \$1.00 per cu. yd. (Wheeler, *Rep. Chf. Engrs.* 1895.)

mixture (the value of f in the formula $M = \frac{1}{6}fbh^2$, where M = bending moment, b = breadth, and h = depth of the beam) is about 500 lbs. per square inch at the age of 19 months. A mixture of 1 C. : 2 S. : 5 stone is

* The cement is here taken as packed in the barrel, and the sand and stone are taken loose. The dry weights of each are also given in the original tables.

TABLE XXXVII.—COMPOSITION, COST, AND STRENGTH OF PORTLAND-CEMENT-CONCRETE BEAMS 10 INCHES SQUARE, MADE AND TESTED IN CONNECTION WITH THE CONSTRUCTION OF THE ST. MARY'S CANAL LOCKS.

(*Rep. Chf. Engrs. U. S. A.*, 1895, Appendix LL, p. 2924.

Proportionate Amounts for 1 cubic foot of Packed Cement (75 lbs. Natural, 104 lbs. Portland).				Proportionate Amounts for 1 Barrel Packed Cement (280 lbs. Natural, 380 lbs. Portland).				Cost of Materials.		Transverse Strength 4-ft. Span.			Transverse Strength at later date, 20-in. Span.		Remarks.
Sand, Loose, Dry. (Cubic Feet.)	Stone, Loose. (Cubic Feet.)	Mortar, Made. (Cubic Feet.)	Rammed Concrete, Made. (Cubic Feet.)	Sand, Loose, Dry. (Cubic Feet.)	Stone, Loose. (Cubic Feet.)	Mortar, Made. (Cubic Feet.)	Rammed Concrete, Made. (Cubic Feet.)	Mortar.	Concrete.	Age when Broken.	Weight for Cubic Feet of Concrete when Broken.	Modulus of Rupture, $f = \frac{3wl}{2Cd^2}$. (Lbs. per sq. in.)	Age.	Modulus of Rupture, $f = \frac{3wl}{2Cd^2}$. (Lbs. per sq. in.)	
1.24	3.0				11.25					2 yrs	158	597	30 m	677	Limestone. Large proportion rather fine
1.67	4.0				15.0					2 yrs	155	551	30 m	627	
1.67	4.0				15.0					6 m	155	465	30 m	656	Do. Tensile strength of mortar 6 mos. in air = 524 lbs.
3.14	7.61	3.76	8.08	11.5	27.8	13.8	29.8	6.42	$4.00	1 yr	144	219	22 m	403	Sandstone ⅜″ to 2½″ and gravel ¼″ to 1½″
3.14	7.61			11.5	27.8				4.00	1 yr	151	239	22 m	408	Limestone and gravel
3.14	7.61			11.5	27.8					1 yr	150	192	22 m	358	Gravel alone
3.12	9.52	3.69		11.4	34.8	13.5		6.42		1 yr	139	139	22 m	319	" "
3.14	7.61			11.5	27.8				4.05	1 yr	143	185	22 m	345	Limestone alone
3.12	9.52			11.4	34.8				3.85	1 yr	141	169	22 m	338	" "
3.07	9.52		8.79	11.2	34.8		32.1		3.85	1 yr	144	283	22 m	461	Limestone ⅜″ to 1″
3.08	7.61		7.90	11.2	27.8		28.9	6.72	4.05	1 yr	148	422	22 m	667	" "
3.08	6.34	4.07	6.95	11.3	23.2	14.9	25.5	5.75	4.30	1 yr	148	374	22 m	591	" "
3.07	9.52	3.64		11.2	34.8	13.3		6.54	3.85	1 yr	143	285	22 m	420	" "
3.08	7.61	3.58		11.2	27.8	13.1		6.72	4.05	1 yr	139	279	22 m	475	" "
3.18	11.42	3.59	10.1	11.6	41.7	13.1	36.8	6.51	3.55	1 yr	145	247	22 m	363	" "
3.18	6.34	3.68	7.28	11.6	23.2	13.5	26.6	6.33	4.10	1 yr	140	319	22 m	587	" "
3.18	11.42	3.74	10.5	11.6	41.7	13.7	38.3	6.22	3.40	1 yr	141	298	22 m	502	" " with 13% screenings
4.16	13.9	4.71	13.40	15.2	50.7	17.2	48.9	5.14	2.85	1 yr	133	177			Sandstone, mixed quite dry

4.16	18.9	4.71	13.40	15.2	50.7	17.2	48.9	5.14	2.85	1 yr	132	213	18 m	229	Sandstone, mixed quite dry
4.10	18.9	4.71	13.40	15.2	50 7	17.2	48.9	5.14	2.85	1 yr	132	204	18 m	227	" "
5.2	18.2	5.72	13.60	19 0	4.81	21.0	49.7	4.30	2.77	1 yr	135	193	18 m	208	" "
5 2	18.2	5.72	13.60	19.0	4.81	21.0	49.7	4.30	2.77	1 yr	141	221	18 m	284	" "
2.1	11.1	2.73	10.47	7 6	40.5	10.0	38.2	8.50	3.27	1 yr	132	255	18 m	302	" very dry
2.1	11.1	2.73	10.47	7 6	40.5	10.0	38.2	8.50	3.27	1 yr	134	269	18 m	382	" somewhat dry
4.16	12.4	4.66	12.32	15.2	45.4	17.0	45.0	5.18	2 94	20 m		357			" "
4.16	12.4	4.66	12.32	15.2	45.4	17.0	45.0	5.18	2.94	20 m		288			" "
3.12	12.4	3.60	11.71	11.4	45.4	13.2	42.8	6.59	3.06	20 m		297			" "
3.12	12.4	3.60	11.71	11.4	45.4	13.2	42.8	6.59	3.06	20 m		351			" "
2.08	12.4	2.70	11 41	7 6	45.4	9 9	41.6	8.50	3.08	20 m		326			" "
2.08	12.4	2 70	11.41	7 6	45.4	9.9	41.6	8.50	3.08	20 m		345			" "
1.04	12.4	1.79	10.64	3.8	45.4	6.5	38 8	12.53	3.24	20 m		310			" "
1.04	12.4	1.79	10.64	3.8	45.4	6.5	38.8	12.53	3.24	20 m		288			" "
0 00	2 09	1.00	2.12	0.0	7.62	3.65	7.73	12.11	11.43	19 m		582			"
0.00	2.09	1.00	2.12	0.0	7.62	3 65	7.73	12.11	11.43	19 m		605			"
1.04	3.76	1.80	3 83	3.8	18.7	6 6	14.0	12.58	6.88	19 m		652			"
1.04	3.76	1.80	3.83	3.8	18.7	6.6	14.0	12.58	6.88	19 m		727			"
2.08	5.57	2.67	5.82	7.6	20.8	9.75	21 2	8.64	4.90	19 m		488			"
2.08	5.57	2.67	5.82	7.6	20.8	9.75	21.2	8 64	4.90	19 m		588			"
3.12	7.71	3 70	8 08	11.4	28.1	13.5	29.5	6.45	3.91	19 m		513			"
3.12	7.71	3.70	8.08	11.4	28 1	13.5	29 5	6.45	3.91	19 m		465			"
4.16	9.86	4 73	10 11	15.2	36.0	17.3	36 9	5.16	3.37	19 m		376			"
4 16	9 86	4.73	10.11	15.2	36.0	17.3	36.9	5.16	3.37	19 m		382			"
5 20	11.93	5.73	12.23	19 0	43.5	20.9	44.6	4.35	3.02	19 m		285			"
5 20	11.93	5.73	12.23	19 0	43.5	20 9	44.6	4 35	3.02	19 m		283			"
6.24	13.80	6 61	14.70	22.8	50.4	24.1	53.7	3.83	2.67	19 m		288			"
6.24	13 80	6.61	14.70	22 8	50.4	24.1	53.7	3.83	2.67	19 m		287			"
2.08	5 64			7.59	20.57					8 m	131	68			" temp. 24° F., stone frozen
2 08	5.64			7.59	20.57					16 m		159			" "
2.08	5.64									16 m		202			Sandstone, temp. 6° to 12° F., stone
2.08	5.64									8 m	146	110	14 m	199	and water cold
2 08	5.64									16 m		245			Sandstone, temp. 6° to 12° F., water
2 08	5.64									8 m	143	145			100° F.
2.08	5.64									8 m	138	142			Sandstone, temp. 6° to 12° F., water
2.08	5.64									16 m		227			156° F.
2.08	5.64									8 m	142	131	14 m	211	Sandstone, temp. 3° to 20° F., stone
2.08	5.64									16 m		223			and water cold
2.08	5.64									8 m	144	242	14 m	340	Sandstone, temp. 3° to 20° F., 18¾%
2.08	5.64									16 m		357			salt in water
2.08	5.64									8 m	145	203	14 m	281	Sandstone, temp. 3° to 20° F., 12¾%
2.08	5.64			7.59	20.57					16 m		273			salt in water
2.08	5.85									11 m		130			Limestone. Shovelled fresh into
2.08	5.85									11 m		128			running water
2.08	6.85									11 m		130			Limestone. Shovelled into running
2.08	5.85									11 m		216			water after 3 hours
2.08	5.85									11 m		130			Limestone. Shovelled into running water after 5 hours
2.08	5.85									11 m		400			Limestone. Made in air and then
2.08	5.85									11 m		435			placed in 8″ water

TABLE XXXVII*a*.—COMPOSITION, COST, AND STRENGTH OF NATURAL-CEMENT CONCRETE BEAMS 10 INCHES SQUARE (ST. MARY'S CANAL LOCKS).

Proportionate Amounts for 1 Cubic Foot of Packed Cement (75 lbs. Natural, 104 lbs. Portland).				Proportionate Amounts for 1 bbl. Packed Cement (280 lbs. Natural, 380 lbs. Portland).				Cost per Cubic Yard.		Transverse Strength on 4-foot Span.			Transverse Strength on 20-inch Span.		Remarks.
Sand, Loose, Dry. (Cubic Feet.)	Stone, Loose. (Cubic Feet.)	Mortar, Made. (Cubic Feet.)	Rammed Concrete, Made. (Cubic Feet.)	Sand, Loose, Dry. (Cubic Feet.)	Stone, Loose. (Cubic Feet.)	Mortar, Made. (Cubic Feet.)	Rammed Concrete, Made. (Cubic Feet.)	Mortar.	Concrete.	Age when Broken.	Weight.	Modulus of Rupture, $f = \frac{3wl}{2Cd^2}$. (Lbs. per Sq. Inch.)	Age.	Modulus of Rupture, $f = \frac{3wl}{2Cd^2}$. (Lbs. per Sq. Inch.)	
1.78	4.18			6.7	15.6					1 yr	136	124			Sandstone ⅜″ to 3″ in size
1.78	4.18			6.7	15.6					1 yr	148	150			Limestone, (D. I.), ⅜″ in to 3″ in size
1.78	4.18			6.7	15.6	9.1		4.14		1 yr	140	222			Limestone (K.I.), "shavings" being flat spalls
2.16	4.18			8.1	15.6	9.0		4.00		1 yr	140	94			Sandstone
2.16	4.18			8.1	15 6					1 yr	141	96			Limestone (D. I.)
2.16	4.18			8.1	15.6					1 yr	139	202	35m	337	Limestone (K.I.), "shavings" or flat spalls
2 30	8 23	2.83	8.44	8.57	3.07	10.6	31 5	3.62	2.25	1 yr	150	120	22m	187	Gravel ¼″ to 1″ in size
2 27	6.86	2.71	7.18	8.47	2.56	10.1	26.9	3.89	2.45	1 yr	151	74	22m	197	" "
2.25	10.17	2 72	9.92	8.42	3.80	10.2	37.1	3.89	2 10	1 yr	146	110	22m	197	" "
2.27	6.86	2.71	6.86	8.47	2.56	10.1	25.6	3 96	2 55	1 yr	146	128	22m	253	Sandstone, ⅜″ to 3″ in size
2 25	10.17	2.72	9.12	8.42	3.80	10.2	34.1	4 24	2.30	1 yr	131	74	22m	151	" "
1.87	5 33	2.42	5.55	7.00	20.0	9.1	20.8	4.24	2.80	1 yr	138	181	18m	276	" " quite dry
1.87	5.33	2.42	5.55	7.00	20.0	9.1	20.8	4.24	2.80	1 yr	139	214	18m	246	" " "
1.87	5.33	2.42	5.55	7.00	20.0	9 1	20 8	4.24	2.80	1 yr	138	175	18m	287	" " "
1.50	5.38	2.07	5.42	5.6	20.2	7.7	20.3	4.88	2.85	1 yr	140	194	18m	261	Sandstone, ⅜″ to 3″ in size, somewhat dry
1.50	5.38	2.07	5.42	5.6	20.2	7.7	20 3	4.88	2.85	1 yr	136	210	18m	256	
1.50	5.38	2.07	5.42	5.6	20.2	7.7	20 3	4 88	2 85	1 yr					
1.12	4.15	1.80	4.60	4.2	17.8	6 7	17.2	5.50	3.17	1 yr	140	275	18m	312	
1.12	4.15	1.80	4.60	4.2	17.8	6.7	17.2	5 50	3.17	1 yr	137	306	18m	422	
0.7̄5	3.05	1.46	3.17	2.8	11.4	5 5	11.9	6 64	4.05	19m		363			Sandstone, ⅜″ to 3″ in size, somewhat wet
0.7̄5	3.05	1.46	3.17	2.8	11.4	5.5	11.6	6.64	4.05	19m		477			
1.50	4.06	2.09	4.37	5.6	15 2	7.8	16 4	4.85	3 20	19m		313			Sandstone, ⅜″ to 3″ in size, somewhat dry
1.50	4.06	2 09	4.37	5.6	15 2	7.8	16.4	4.85	3.20	19m		351			
2.25	5.90	2.85	6.00	8.4	22.1	10 7	22 5	3.65	2.70	19m		206			
2.25	5.90	2 85	6.00	8.4	22 1	10 7	22.5	3 65	2.70	19m		274			
3 00	7.40	3.55	7.60	11.2	27.7	13.3	28.5	3.10	2.40	19m		187			
3.00	7.40	3.55	7.60	11.2	27.7	13 3	28.5	3 10	2 40	19m		185			
2 25	6.56	2.85	6 30	8.4	24.5	10 7	23 6	3.63	2 70	11m		237			Limestone (D. I.), no screenings
2.25	6.56	2.85	6.30	8.4	24 5	10.7	23.6	3.63	2.70	11m		229			
2 28	6.56	2 85	6.56	8.5	24.5	10 7	24.6	3 63	2 60	11m		177			Limestone (D. I.), 10 pts. screenings to 100 pts stone
2 28	6.56	2 85	6 56	8 5	24.5	10.7	24.6	3.63	2 60	11m		216			
2 2[illegible]	6.56		6 73	8 4	24.5		25 2		2.50	11m		216			Limestone (D. I.), 17 pts. screenings to 100 pts. stone
2 25	6.56		6 73	8 4	24.5		2[illegible] 2		2.50	11m		173			
1 29	7.30	2 02	6.82	4.83	27 3	7 56	25 6	5 00	2.55	11m		230			
1.29	7.30	2.02	6 82	4.83	27.3	7.56	25 6	5.00	2.55	11m		225			
2 28	6 56	2 89	6.82	8 5	24 5	10 85	25.6	3.57	2.50	11m		187			Limestone (D. I.), 50 pts. screenings to 100 pts. stone
2.28	6.56	2.89	6 82	8.5	24.5	10 85	25 6	3.57	2.50	11m		216			
2 28	6 56	2 85	7 13	8 5	24.5	10 7	26 7	3 63	2.35	11m		101			Limestone (D. I.), 100 pts. screenings to 100 pts. stone
2 28	6 56	2.85	7.13	8.5	24.5	10 7	26.7	3 63	2 35	11m		144			
2 28	6.56	2.85	6 82	8 5	24 5	10 7	25 6	3.63	2.50	11m		130			Limestone (D I.), screenings only

25 per cent stronger but 12 per cent more expensive for a given strength, while a mixture of 1 C. : 4 S. : 10 stone is 12 per cent weaker and about 10 per cent more expensive for a given strength, than the mixture 1 C. : 3 S. : 7½ stone, all by volume.

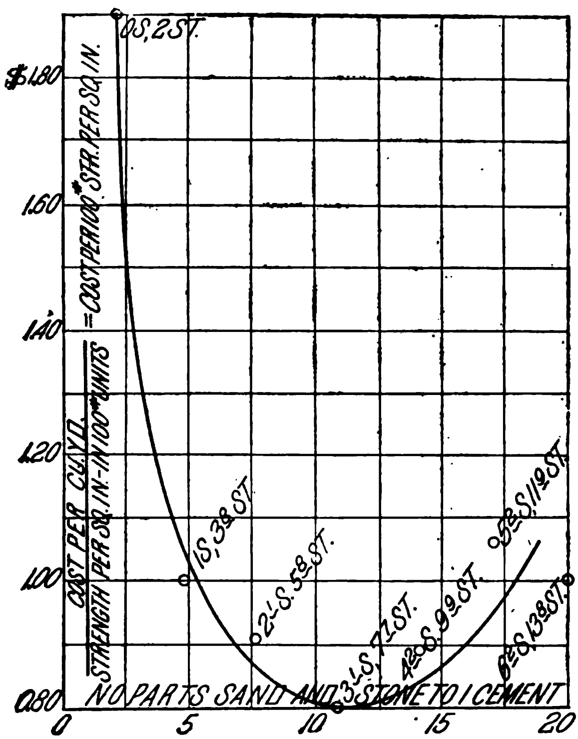


Fig. 552.—Economy in Portland-cement-concrete Mixtures as shown by Tests of Concrete Beams. Each result the mean of two tests on beams 10 in. square. Cement $3.00 per bbl.; sand $0.50 and stone $1.00 per cu. yd. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2926.)

In Fig. 553 are shown the results of tests on Portland-cement concrete beams where the mortar was always the same (1 C. : 3 S.), while the stone ingredient varied. Results are here shown for an age of one year and also of 22 months. It is very noticeable that the strength at 22 months is much greater (about 60 per cent greater) than at 12 months. This apparently great increase in strength may be partly due to the shorter length of beam, this being but 20 inches between supports, as compared with 48 inches for the 12-month tests.

Here again the maximum economy is found for the mixture 1 C. : 3 S. : 7½ stone, as shown by the curve marked "Ratio of cost to strength." For this mixture the voids are just filled, while with a less amount of stone there is an excess of mortar, and with a greater proportion the voids are not filled.

In the case of natural-cement mortar, the cost of the cement being now $1.30 per barrel instead of $3.00, the most economical mixture is about 1 C. : 1½ S. : 4 stone, as shown by Fig. 554. The data on these mixtures were as follows:

Mixture.	Cost per Cubic Yard in Dollars.	Cross-breaking Modulus in Pounds per Square Inch.
1 C. : ¾ S. : 3 Stone.............	4.05	420
1 C. : 1½ S. : 4 Stone.............	3.20	332
1 C. : 2½ S. : 5.9 Stone...........	2.70	240
1 C. : 3 S. : 7.4 Stone..........	2.40	186

The effect of making a portion (13 per cent) of the broken stone consist of screenings, such as are formed when a rock-crusher is employed, is shown in Fig. 553. This is very marked where there is a deficiency of mortar.

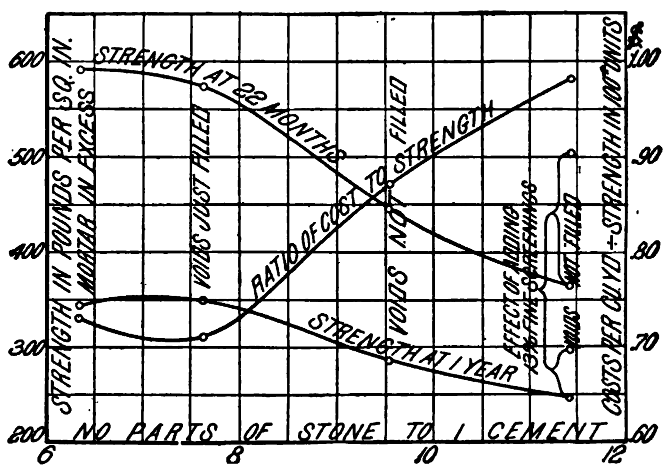


FIG. 553.—Strength of Portland-cement-concrete Beams. Effect of Varying Proportions of Stone to 1 of Cement, always using 3 Sand to 1 Cement. Each result is the mean of two tests on beams 10 in. square. Stone passed 1-in. screen and stopped on ¾-inch. Cement $3.00 per bbl.; sand $0.50 and stone $1.00 per cu. yd. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2924.)

Thus with the mixture 1 C. : 3 S. : 11.4 stone the strength was increased about 25 per cent by making the broken stone consist of 13 per cent screenings. Where the mortar is sufficient to fill the voids of the stone or gravel the fine screenings would weaken the concrete, as they would be equivalent to so much additional sand.

420. Filtration through Concrete.—In Fig. 555 are given results of filtration experiments on Portland-cement concrete of different mixtures. The most remarkable feature of this diagram consists in the evidence it offers of the rapid closing of the openings in the mass. At the end of 18 days the

filtration had practically ceased in all the mixtures, although the concrete was three months old when the experiments began. Whether this rapid diminution in the rate of filtration is due to the progressive crystallization of the cement as a result of the flow of water through it or from some other cause does not appear. It is commonly accepted that the disintegration of Portland-cement concrete is primarily due to its permeability, and hence filtration tests are made to determine this property. As all of the mixtures shown in Fig. 555 become practically impervious to water in a few days, they should be considered as entirely satisfactory on this score.

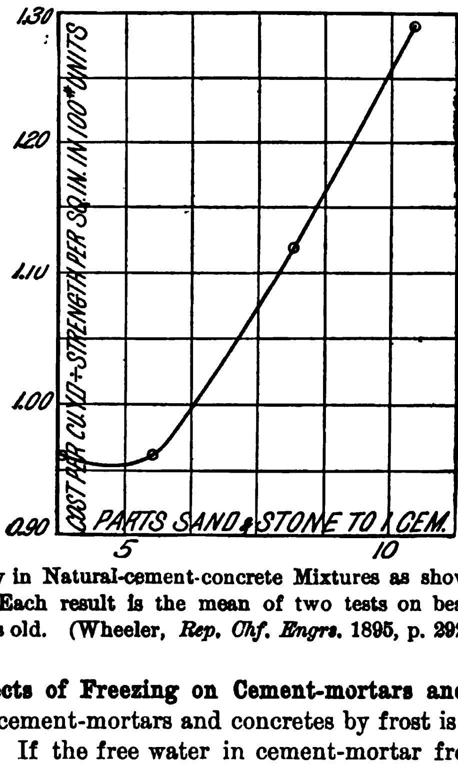


Fig. 554.—Economy in Natural-cement-concrete Mixtures as shown by Tests of Concrete Beams. Each result is the mean of two tests on beams 10 in. square and nineteen months old. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2929.)

421. The Effects of Freezing on Cement-mortars and Concretes.—The disintegration of cement-mortars and concretes by frost is due to the expansive force of ice. If the free water in cement-mortar freezes before it becomes combined by crystallization in the process of hardening, evidently this mortar cannot set or harden until the ice melts. But when the temperature is low or near that of freezing the hardening action is very small, so that the mortar is likely to dry out before the water present is taken up by the hardening cement. In this case it will never harden, and this is apt to be the case with the outer or exposed portions of cement-masonry. Again, when the cement has set and partially hardened, if the freezing of the remaining water (or of that which the porosity of the mortar allows to enter it from without) produces an expansive force in excess of the cohesive strength of the mortar at the time, then the bond is broken by the expanding ice, and

on thawing out the mortar crumbles from the disintegrating action of the frost, the same as a soft, porous stone or brick will do. Portland-cement mortar being stronger than that made with natural cement, it resists this disintegrating action better, and hence the general assumption that Portland cement may be used in freezing weather and natural cement may not.

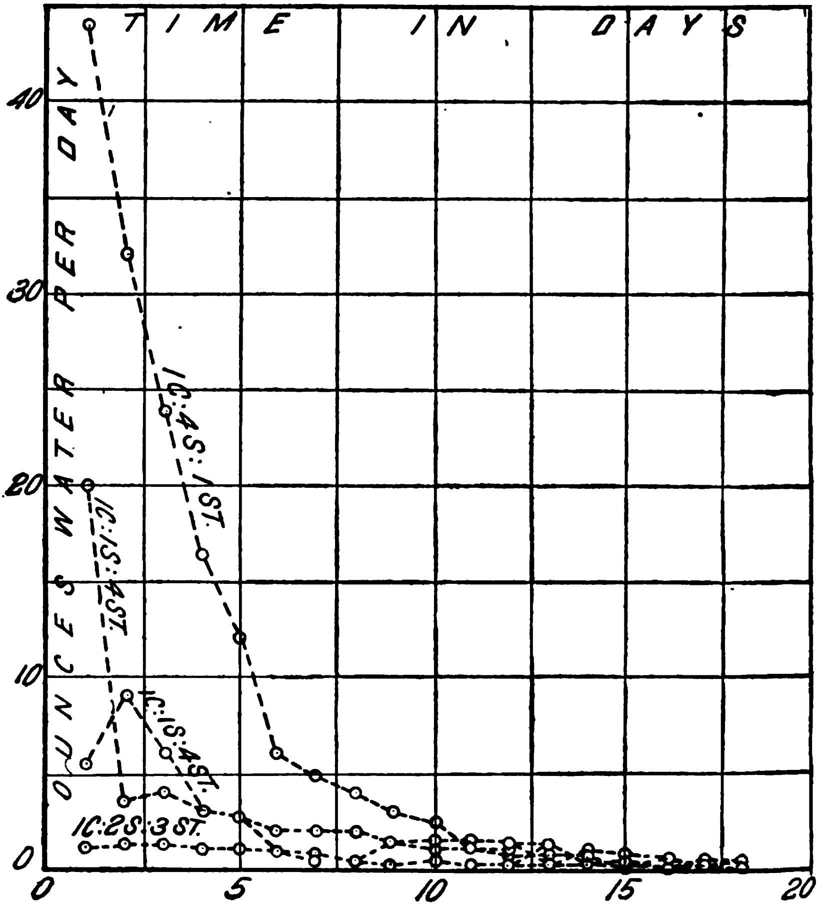


FIG. 555.—Filtration of Sea-water through 1 cu. ft. of Portland-cement Concrete, Three Months Old, under 24 ft. head. (*Inst. Civ. Engrs.*, vol. CVII. p. 95.)

The Universal Rule to be followed when using cement in freezing weather is to use the minimum amount of water in gauging the mortar, to keep it from freezing until it has acquired a considerable strength, and to protect it from the weather or from the action of alternate wettings and freezings. It should also be made richer in cement than that used at ordinary temperatures. There are various ways of preventing freezing, as, for example:

1. Warming the water or sand or stone, or all of them, as occasion seems to require.

2. In very cold weather, in addition to the provisions in (1), the work may be covered with earth or manure, or housed and a fire maintained, etc.

3. In place of these methods of maintaining a temperature above the freezing-point, the water may be dosed with salt or glycerine or alcohol, until it will not freeze at the temperatures anticipated.

All of these methods are used and all are satisfactory. The cheapest and most common method is to make a brine of the water used in gauging the mortar. In. Fig. 556 the proper percentages of salt, glycerine, and alcohol

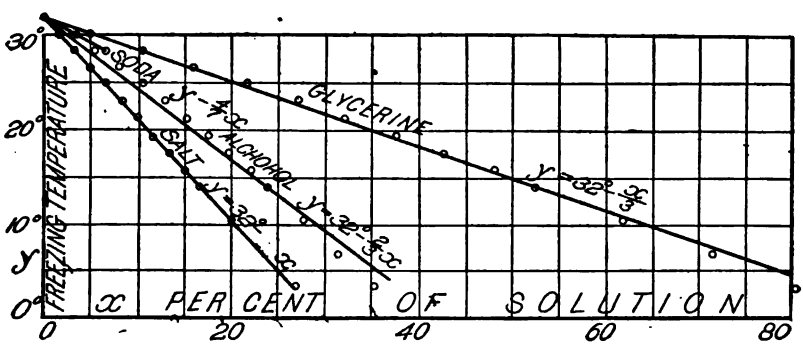


FIG. 556.—Effect on the Freezing-point of Cement of Various Proportions of Glycerine, Alcohol, and Salt. (Tetmajer, vol. VII. p. 85.)

are shown to prevent freezing at the various temperatures from 32° to 0° F. From this it appears that salt is the most efficient agent as well as the cheapest. From this diagram we have, approximately,

No. degrees F. freezing temp. is reduced = per cent salt used.

Thus if it is assumed that the temperature will not fall below 22° F., then 10 per cent (by weight) of salt should be added to the water. If a temperature of 10° F. is to be provided for, use 22 per cent of salt. Doubtless a less proportion of salt would prove effective at these temperatures, especially with concrete in large masses, as the chemical reactions which accompany the hardening of the cement develops a considerable amount of sensible heat. (See Art. 312 and Figs. 333 and 333*a*.)

Any cement sets very much slower at a low temperature than at a higher, as is shown by Fig. 557, which is complementary to Fig. 333, p. 414. Thus a mortar which will set completely (by the method of testing employed) in four hours at a temperature of 35° C. (95° F.) would require twenty hours at 5° C. (41° F.) and thirty-eight hours at 0° C. (32° F.). In estimating the time required for setting, therefore, this temperature-effect must be allowed for.

In Fig. 558, upper half, is shown the effect of salt on tension briquettes

of Portland cement which were moulded in a room where the temperature was 8° F. (24° F. below freezing), and where the briquettes were frozen hard in half an hour, and remained frozen 60 days. They remained in the open air and hardened when they thawed out. It should be noted that the briquettes grew weaker between the ages of 6½ and 9½ months.

In the lower half of this figure are given the results of tests of the same cement-mortar moulded in air at a temperature of 21° F., and left frozen for three days and then placed under water for the remaining period.

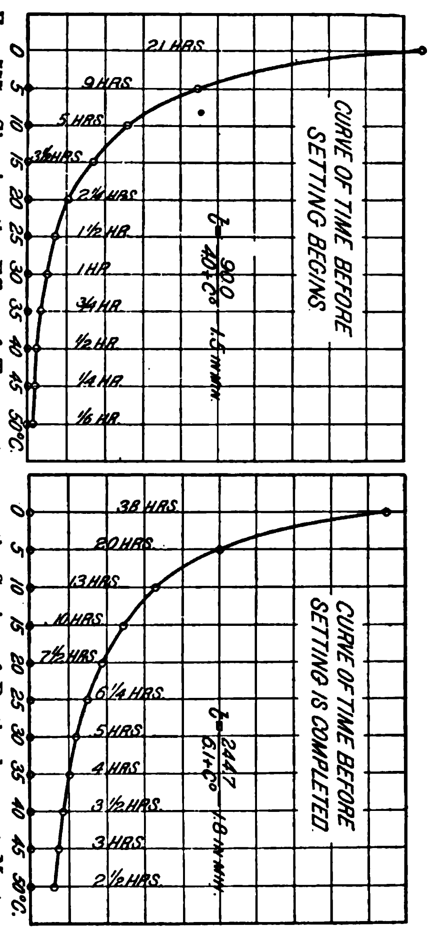


Fig. 557.—Showing the Effect of Temperature on the Setting of Portland-cement Mortar, 1 C. : 3 S. The temperature is given in degrees centigrade. (Tetmajer's *Communications*, vol. VI, Plate VIII.)

In the former series (hardened in air) we might conclude that more than 5 or 10 per cent of salt weakened the mortar somewhat, while in the latter (hardened in water) the briquettes increased in strength up to 20 per cent salt. This is, furthermore, a more typical case. Cement-masonry is not

likely to be laid in extremely cold weather or when such weather is likely to occur before the cement has set.

The worst set of conditions for cement-mortar to withstand is that of a succession of temperatures below and above the freezing-point. If the mortar freezes as soon as laid, there is no bond to be broken, and no injury can result provided that when it thaws out it remains unfrozen long enough to harden. But if it begins to set and then freezes again before the cohesive strength can resist the expansive force of the frost, then it is cracked and these severed surfaces will never again unite. Water then enters such cracks and further disintegration follows.

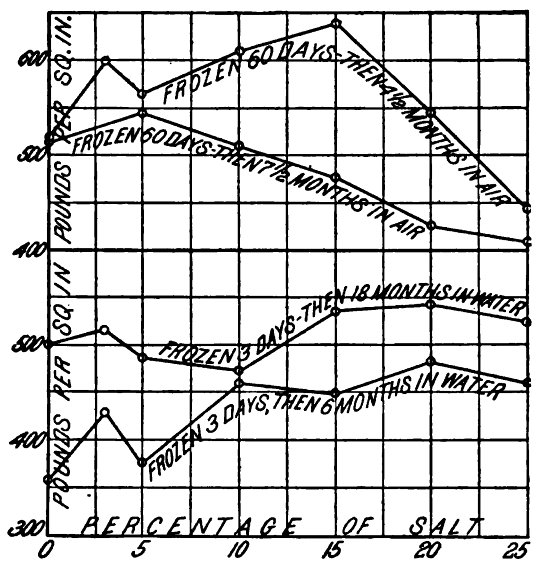


FIG. 558.—Effect of Salt on Portland-cement Mortar, 1 C.: 2 S., made in Freezing Weather. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2968.)

A careful distinction must be drawn between contraction cracks, which may always be found in long masonry walls in cold weather, and disentegration cracks from the expansive force of water freezing on the interior.

In Fig. 559 the effect of salt on the strength of both natural- and Portland-cement mortar is shown. These briquettes were not subjected to freezing temperatures, and hence the results are not very significant. These and similar tests simply determine whether or not the salt weakens the mortar if used at temperatures above freezing. But as it is never used in this way the results are scarcely to the point.

Similar to these are the results shown in Fig. 560, except that all these were made with a saturated solution of salt (about 30 per cent), and with different proportions of sand, up to 2 S. : 1 C., which is as much sand as should be used in freezing weather.

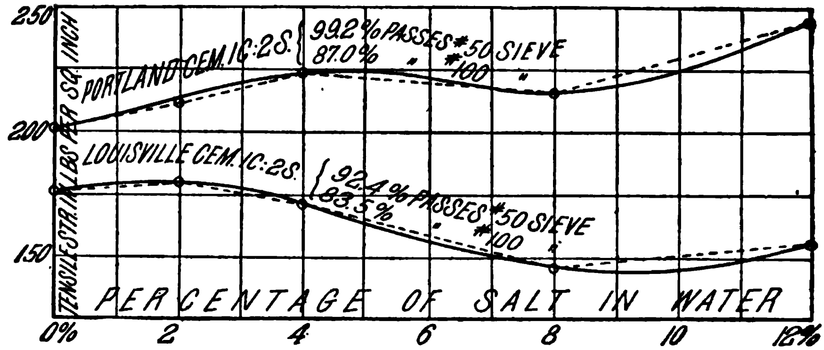


FIG. 559.—Effect of Salt on Strength of Cement-mortar Six Months Old. (*Jour. Assoc. Eng. Socs.*, vol. IX.)

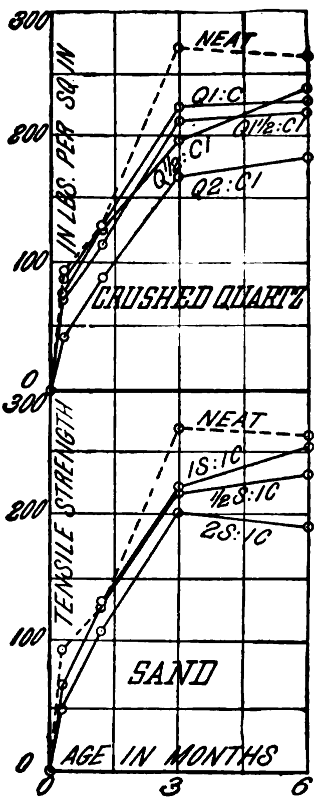


FIG. 560.—Tensile Strength of Quartz and Sand Mortars with a Saturated Solution of Salt. (*R. R. Gazette*, 1892.)

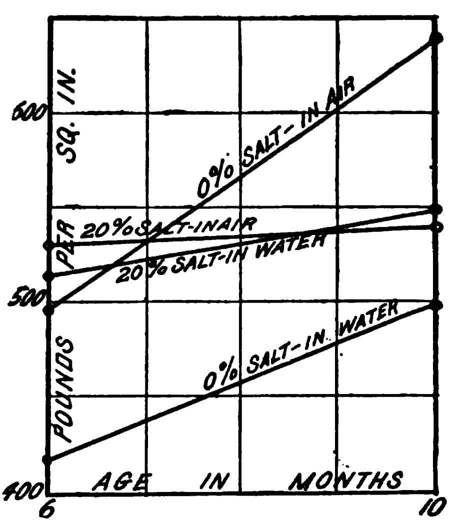


FIG. 561.—Effect of Salt on Portland-cement Mortar, 1 C. : 2 S., made in Freezing Weather. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2971.)

In Fig. 561 we have the extreme conditions of fresh water and a brine containing 20 per cent salt, used in making up Portland-cement-mortar briquettes in a room temperature of 13° to 16° F. (at which the 20-per-cent salt-mixtures would not freeze), and left at this temperature for three days. Then 40 of these briquettes were placed in water and the remaining 40 were stored in the open air. This being early in January at Lake Superior, it is likely the " air " briquettes remained frozen for some months. When they did thaw out, the temperature conditions seem to have been favorable to their hardening. Probably they had so dried out while frozen that there was

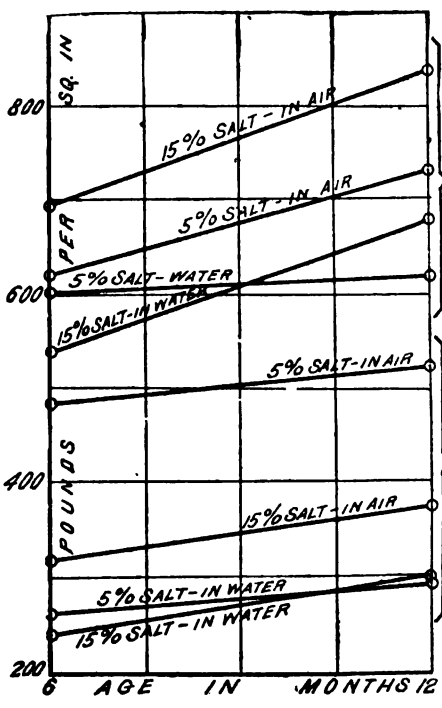


FIG. 562.—Use of Salt in Portland-cement Mortar, 1 C. : 1 S. and 1 C. : 4 S. Those left in air remained frozen about sixty days. Those put in water were first frozen in air three days. (Wheeler, *Rep. Chf. Engrs.* 1895, p. 2970.)

not sensible moisture enough to produce expansion by successive freezing and thawing. It is to be presumed they were not exposed to the weather, but were at least under shelter. It is a common maxim with civil engineers of large experience in such matters that if cement-masonry, laid in freezing weather, remains frozen till dry, or if it " freezes dry," it will harden without injury, but if it freezes and thaws sucessively while yet "green" it will be injured, if not ruined.

In Fig. 562 are shown results on two mixtures, 1 C. : 1 S., and 1 C. : 4 S.

These briquettes were made up at 18° F. (at which temperature the 15-per-cent salt-mixtures would scarcely freeze), and the "air" briquettes put in the "open air" after three days, this being early in January, 1894, at Lake Superior. Even the 15-per-cent-salt briquettes doubtless were

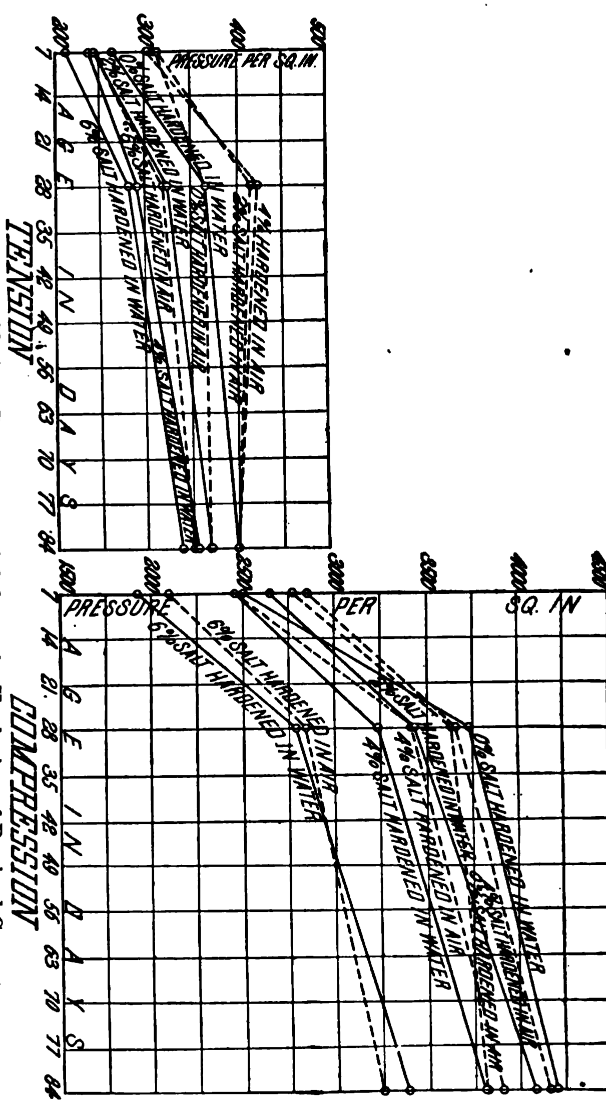


Fig. 563.—Effect of Various Percentages of Salt on the Hardening of Portland Cement. (Tetmajer, vol. VII. p. 84.)

frozen on being put out in the open air. The results are not such as to lead to any positive conclusion.

Prof. von Tetmajer has experimented largely with anti-freezing solutions

for mixing cement-mortars, those of salt on Portland-cement mortars being shown in Fig. 563, and on natural-cement mortars in Fig. 564. All these tests were made at the standard temperature of 65° F., so that they show simply the effect of the salt on the tensile and the compressive strength when hardened in air and under water. In every case any addition of salt weakened the mortar, an addition of 6 per cent of salt reducing the strength

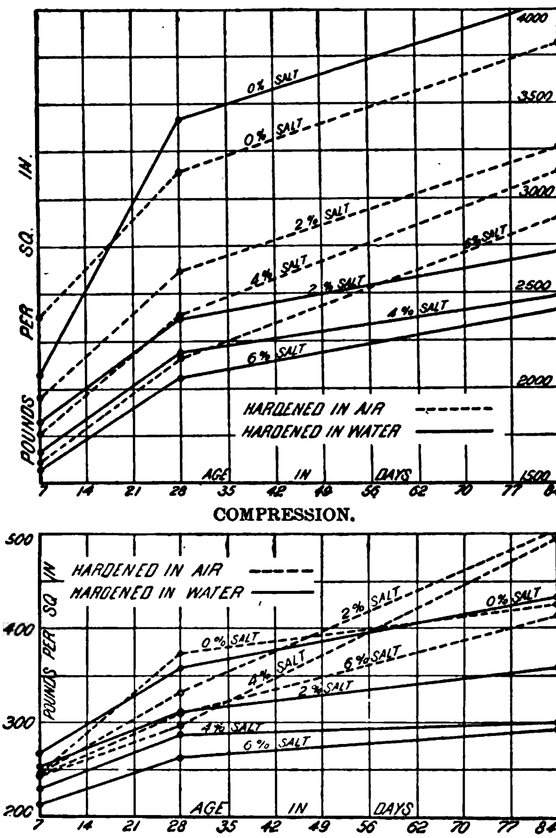


FIG. 564.—Effect of Salt on the Hardening of Natural-cement Mortar 1 C · 3 S. (Tetmajer, vol. VII. p. 34.)

about 25 per cent. The reduction was more marked with the natural- than with the Portland-cement mortars.

In Figs. 565 and 565*a* Tetmajer gives us similar results on Portland-cement mortars of anti-freezing mixtures of solutions of glycerine and of alcohol. In every instance these ingredients also weakened the mortar.

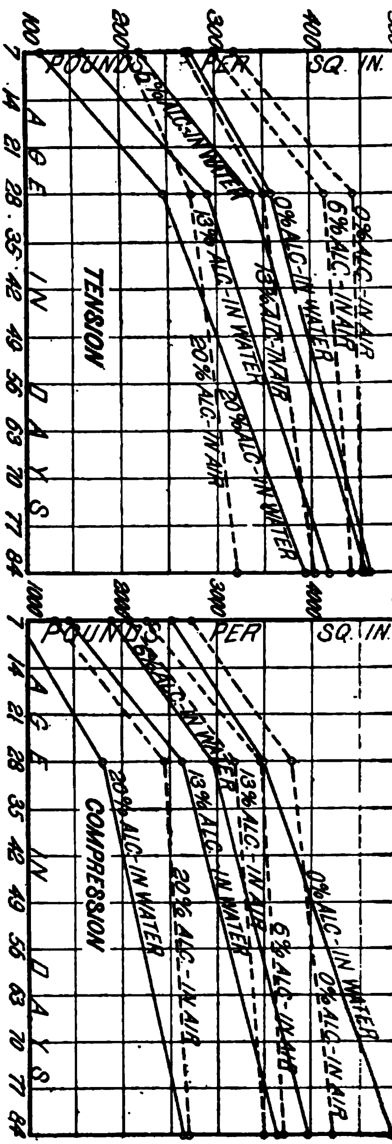
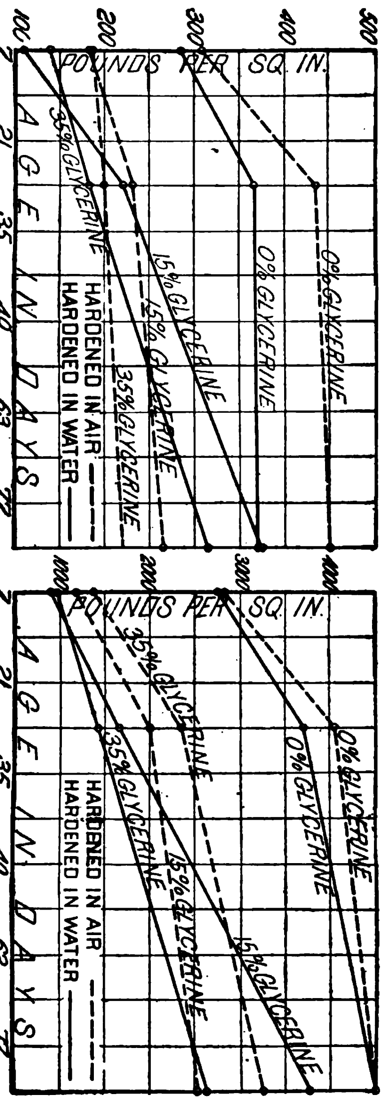


Fig. 565.—Effect of Alcohol in Water used in Mixing and in the Bath on the Strength of Portland-cement Mortar, 1 C. : 3 S.



TENSION. COMPRESSION.

Fig. 565a.—Effect of Glycerine on the Strength of Portland-cement Mortar, 1 C. : 3 S. (Tetmajer, vol. VII. p. 24.)

422. Concrete Mixtures should be so proportioned as to produce as nearly as possible a solid mass with the least proportion of cement. Broken stone is not at all essential to a first-class concrete, a clean gravel serving quite as well. Thus Col. G. H. Mendell gives the following formula with the resulting volumes at each stage*:

	Cubic Feet.
One barrel of Portland cement, measured loose........	4.50
Water added..	1.88
Volume of paste....................................	3.90
Sand equal to three times the volume of paste.........	10.70
Water added..	2.25
Volume of mortar...................................	11.21
Gravel, 3/4 in. and less............................	36.70
Volume of loose concrete............................	44.24
Final volume tamped in place........................	36.20

Here we have 51.9 cubic feet of loose solids finally compacted to a volume of 36.2 cubic feet, or to 69 per cent of the loose volumes when measured separately.

423. Concrete Structures in Sea-water.—On the subject of the permanency of cement-concrete when exposed to the action of sea-water Dr. Michaëlis, the highest possible authority, says† :

"The main points to be considered in erecting permanent structures in sea-water with the aid of hydraulic cements—in other words, concrete—are:

"(1) From the physical point of view, completely impermeable mixtures should be made, composed of one part of cement with two or at the most two and a half parts of sand, of mixed grain, of which at least one third must be very fine sand. To this the requisite quantity of gravel and ballast should be added. This impermeable layer should surround the porous kernel on all sides in sufficient thickness, even underneath. It would, perhaps, be unnecessary waste of material, in the case of thick walls, to use the impermeable mixture throughout; but, so far as possible, the compact shell and the poorer kernel should be made in one operation. Where this is not possible, and the shell is added subsequently, numerous iron ties should be used.

"(2) From the chemical point of view, cements or hydraulic limes rich in silica, and as poor as possible in alumina and ferric oxide, should be used, for aluminate and ferrate of lime are not only decomposed and softened rapidly by sea-water, but they also give rise to the formation of double compounds which in their turn destroy the cohesion of the mass by producing cracks, fissures, and bulges. The salts contained in sea-water, especially the sulphates, are the most dangerous enemies of hydraulic cements. The lime

* In *Jour. Assc. Eng. Socs.*, vol. XIV. p. 248.

† In *Trans. Inst. Civ. Engrs.*, vol. CVII. p. 375.

is either dissolved and carried off by the salts, and the mortar thus loosened, or the sulphuric acid forms with it crystalline compounds as basic sulphate of lime, alumino-sulphate and ferro-sulphate of lime, which are segregated forcibly in the mortar, together with a large quantity of water of crystallization, and a consequent increase in volume results. The separation of hydrate of magnesia is only the visible but completely innocuous sign of these processes. The magnesia does not in any way enter into an injurious reaction with silica, alumina, or ferric oxide; it is only displaced by the lime from its solution in the shape of a flocculent, slimy hydrate which may be rather useful in stopping the pores, but can never cause any strain or expansion, even if it subsequently absorbed carbonic acid.

"The carbonic acid, whether derived from air or water, assists the hydraulic cement as a preservative wherever it comes into contact with the solid mortar. It could only loosen the latter if present in such an excess that bicarbonate of lime might be formed.

"(3) The use of substances which render the mortar, at any rate in its external layers, denser and more capable of resistance. Such substances are:

"(*a*) Sesquicarbonate of ammonia (from gas-liquor) in all cases where long exposure to the air is impossible. Such a solution, applied with a brush or as a spray and then allowed to dry, converts the hydrate of lime into carbonate of lime. The latter is not acted upon by the neutral sulphates present in sea-water. It must be repeated that it is a decidedly erroneous opinion that the texture of otherwise sound cements is injured by the action of carbonic acid; on the contrary, it renders them more capable of resistance, except in the above-mentioned case, which is extremely rare when bicarbonate of lime is formed and goes into solution.

"(*b*) Fluosilicates, among which magnesium fluosilicate is most to be recommended. The free lime is converted into calcium fluoride and silicate of lime, and, in conjunction with the liberated hydrate of magnesia, these new products close the pores of the mortar. Both salts are sufficiently cheap to be used on a large scale.

"(*c*) Last, not least, barium chloride. Two or three per cent of the weight of the cement is dissolved in the water with which the concrete is mixed. This forms perfectly insoluble barium sulphate with the sulphates of the sea-water, while the magnesia remains in solution as magnesium chloride. Although in this case there can be no further closing of the pores, yet the insoluble barium sulphate which is formed affords some protection and does not give rise to any increase of volume (swelling). From two to three per cent of barium chloride does not in any way diminish the strength, as has been proved by means of comparative tests of English and German cements. Frequently the strength of the mortar is increased by this addition. This substance is only to be used in the external, perfectly water-tight skin of the concrete; in other words, in the 4- to 8-inch coating, composed of 1 cement, 1 to 2 sand, and 3 to 4 coarse gravel, flint, broken stone, etc."

TABLE XXXVIII.—TESTS OF THE FIRE-RESISTING QUALITIES OF DIFFERENT KINDS OF CONCRETE.

(Hamburg Commission Report, 1895.)

Number of Test.	Composition of Concrete.	Time of Heating.	Manner of Cooling.	Result of Heating.	Result of Wetting after Heating.	Temperature measured by Pyrometer.
I.	1 part cement, 7 parts river-gravel.	3¾	Suddenly.	Broken.	Crumbled entirely.	Highest temperature 1060° C.
			Slowly.	"	"	"
II.	1 part cement, 8 parts river-gravel.	3¾	Suddenly.	"	"	"
			Slowly.	"	"	"
III.	1 part cement, 3 parts sand, 5 parts broken stone.	3¾	Suddenly.	Not broken, but mortar very tender.	Lost coherence	"
			Slowly.	"	"	"
IV.	1 part cement, 7 parts washed bank-gravel.	3¾	Suddenly.	Showed very little coherence.		"
			Slowly.	"		"
V.	1 part cement, 8 parts washed bank-gravel.	1	Suddenly.	"	Crumbled.	After 1 hour 780° C.
			Slowly.	"	"	"
VI.	1 part cement, 7 parts fine cinder.	1¼	Suddenly.	Not broken, but broke upon striking.	Showed good coherence; did not suffer.	After 1¼ hours 780° C.
			Slowly.	"	"	Highest temperature 1060° C.
VII.	1 part cement, 8 parts fine cinder.	3¾	Suddenly.	"	"	After 1¼ hours 780° C.
			Slowly.	"	"	Highest temperature 1060° C.
VIII.	1 part cement, 7 parts coarse cinder.	3¾	Suddenly.	Not broken; showed relatively the highest degree of coherence, particularly in the centre.	Did not suffer.	"
			Slowly.	"	"	"
IX.	1 part cement, 8 parts coarse cinder.	3½	Suddenly.	"	"	Highest temperature 940° C.
			Slowly.	"	Did not suffer; friable edges.	"
X.	1 part cement, 3 parts sand, 5 parts broken basalt.	3½	Suddenly.	Not broken, but broke upon striking.	Crumbled.	"
			Slowly.	Broken.	Coherence very slight.	"
XI.	1 part cement, 7 parts sand.	3½	Suddenly.	Broken in 3 pieces.	Low degree of coherence.	"
			Slowly.	Broken; very tender.		"
XII.	3 parts Trass-mortar,* 5 parts cinder	3½	Suddenly.	Broke in taking out.	Crumbled completely.	"
			Slowly.	Broken; very tender.		"
XIII.	1 part Trass, 2 parts slacked lime, 20 parts river-gravel.	3½	Suddenly.	Completely broken.		"
			Slowly.	Crumbled to powder.		"

* Trass-mortar = 1 part Trass, 2 parts slacked lime, 3 parts sand.

TESTS OF THE FIRE-RESISTING QUALITIES OF CONCRETE—*continued.*

Number of Test.	Composition of Concrete.	Time of Heating.	Manner of Cooling.	Result of Heating.	Result of Wetting after Heating.	Temperature measured by Pyrometer.
XIV.	1 part cement, 7 parts pumice-sand.	3½	Suddenly.	Not broken; showed some coherence, particularly in centre.	Outer part tender, and pieces fell off.	Highest temperature 940° C.
			Slowly.	Not broken.	Friable on edges.	"
XV.	1 part cement, 3 parts sand, 5 parts grit-stone.	3½	Suddenly.	Fell to pieces upon touching.	Crumbled entirely.	"
			Slowly.	Broken. Coherence almost completely lost.		"
XVI.	7 courses of brick (one brick deep) in cement-mortar *	3½	Suddenly.	Mortar very tender and lost its binding power; some bricks cracked.		"
			Slowly.			"

* Cement-mortar 1 to 3.

424. The Fire-resisting Qualities of Concretes.—Since concrete construction is now used very largely in large buildings, its fire-resisting qualities become of supreme importance in such works. The most elaborate investigations ever made into these qualities of various concrete mixtures was carried out by a commission especially appointed by the city of Hamburg, Germany, for this purpose some years ago, and who issued an elaborate report in 1895.* Table XXXVIII embodies their results on fire-tests of sixteen different concrete mixtures.

It will be observed that all the sand, gravel, stone, and fine-cinder concretes failed to stand the test. Only the coarse-cinder concrete (1 C. to 7 or 8 cinder) gave good results. Even the wetting while hot did not affect it. It would seem, therefore, that a screened-cinder concrete would give excellent results.

Hydraulic cements, both natural and Portland, harden by a process of crystallization, requiring the combination with an amount of water equal to some 15 per cent of the weight of the pure cement. When this crystallized mass is highly heated the water of crystallization is driven off and the cement is reduced to an inert mass or powder. A high heat long continued, therefore, is fatal to the strength of all cement mortars and concretes.

Mr. J. S. Dobie has found † that neat Portland-cement briquettes two months old, gradually heated to 1000° F. and then removed from the furnace and allowed to cool in the air, lose about 10 per cent of their weight

* Results of tests upon various kinds of patented heat-insulating systems of protecting the iron framework of a building are also given in this report.

† In the *Digest of Physical Tests*, vol. I. p. 212 (1896).

and 50 per cent of their tensile strength. If heated suddenly to 1775° F. and cooled in the air, they lost 10 per cent of their weight and 80 per cent of their strength, the results in both cases, however, being far from uniform. When plunged in water on removing them from the furnace, they fell to pieces in both instances.

Mr. T. T. Johnston has shown* that for natural- and Portland-cement briquettes, both neat and 1 C. : 1 S., heated to a dull red after thorough drying, gave losses of strength as follows:

LOSS OF STRENGTH OF CEMENT-MORTAR FROM HEATING.

Kind of Cement.	Neat.	Mortar.
Natural cement	89 per cent	(1 C. : 1 S.) 61 per cent
Portland cement................	58 " "	(1 C. : 3 S.) 70 " "

It is evident, therefore, that fire-proof construction should not rely on a sand or stone cement-concrete for tensile strength. If it be used only in compression, a metal base resisting the tensile deformations, then it may be able to carry its load during the fire, but it would probably require reconstruction afterwards, especially if water reached the cement portions while they were highly heated.

425. Portland-cement Cinder-concretes.—The strength, weight, cost, and economy of a Portland-cement cinder-concrete construction in St. Louis in 1896, taking actual prices for large buildings and adding 5.5 cts. per cubic foot for laying, are shown graphically in Fig. 566. The test mixtures were made without special care, and had therefore much less strength than the same ingredients would have in practice if mechanically mixed.† The mixtures are arranged in the diagrams in the order of their economic values, that is, in the order of their rank in the quality, $\frac{\text{strength}}{\text{cost}}$. The specimens were all 6 in. square and 12 in. high, and were crushed in the direction of the longest dimension. They were all 30 days old when tested. The cinders were the ordinary furnace product of St. Louis, obtained by burning the Illinois bituminous coal under boilers. It is mostly a fine ash with considerable unburned coal in its composition.

It will be observed that the mixture 1 cement : 1 sand : 3 cinder is at once both the most economical, $\left(\frac{\text{strength}}{\text{cost}} = \text{max.},\right)$ and also the strongest for its

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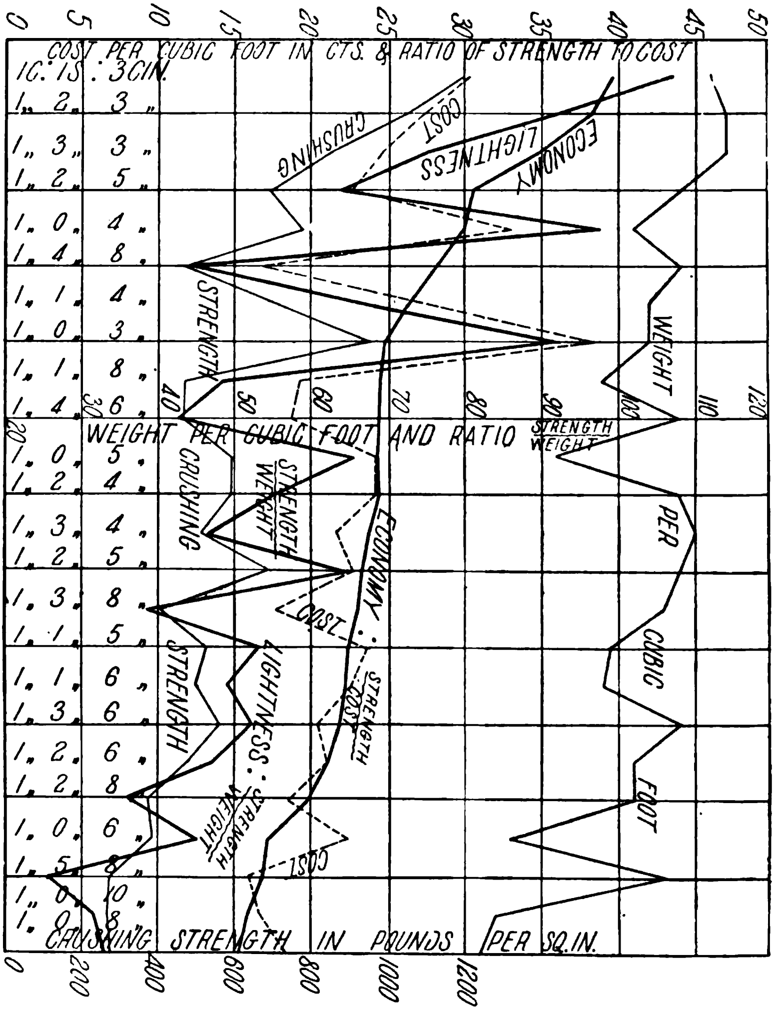
* *Engineering Record*, vol. xxxv. p. 54.

† The specimens were prepared under the direction of Mr. A. L. Johnson, Assoc. M. Am. Soc. C. E., and were tested by the author.

weight, $\left(\frac{\text{strength}}{\text{weight}} = \text{max.}\right)$ It would seem, therefore, that where quantity is proportioned to obtain a given strength, this would be the mixture to employ if cinder-concrete were to be used.



COMPOSITION OF CONCRETE MIXTURES.

FIG. 566.—Portland-cement Cinder-concrete. Strength, Weight, Cost, and Economy of, for St. Louis, 1896. Tests made by the Author.

TABLE XXXIX.—CROSS-BENDING STRENGTH OF PORTLAND-CEMENT CINDER-CONCRETE MIXTURES WITH AND WITHOUT EXPANDED METAL BASE.

Slabs 36 in. long, 12 in. wide, and 4 in. thick tested as beams on supports 32 in. apart. (Author's Records.)

Mixture.	With or Without Expanded Metal Base.	Modulus of Elasticity, Pounds per Square Inch.	Modulus of Rupture, $f=\frac{3}{2}\frac{wl}{bh^2}$, Pounds per Square Inch	Modulus of Strength at the Apparent Elastic Limit, Pounds per Square Inch.	Total Deflection under Maximum Load, in Inches.
1 cement : 5 cinder	With	970,000	150	150	0.008
1 " : 5 "	Without	820,000	550	300	.450
1 " : 6 "	Without	490,000	150	150	.015
1 " : 6 "	With	930,000	450	200	.100
1 cement : 1 sand : 1 cinder	Without	690,000	170	150	.015
1 " : 1 " : 1 "	With	1,930,000	465	200	.130
1 " : 2 " : 5 "	Without	430,000	100	100	.002
1 " : 2 " : 5 "	With	800,000	575	300	.169
1 " : 3 " : 5 "	Without	490,000	88	75	.013
1 " : 3 " : 5 "	With	510,000	370	200	.142
1 " : 1 " : 6 "	Without	260,000	100	75	.020
1 " : 1 " : 6 "	With	540,000	445	150	.158

CHAPTER XXXI.

RESULTS OF TESTS ON STONE AND BRICK.

STONE.*

426. The Structural or Building Stones consist principally of

The Granites (including the Gneisses),
The Limestones (including the Marbles), and
The Sandstones (including Breccias and Conglomerates).

The granites are unstratified, eruptive rocks, and are composed of quartz (pure silica, SiO_2) having a hardness of 7;† of feldspar (silica, alumina, together with potash, soda, or lime) with a hardness of 6; hornblende, hardness 5 to 6; and small scales of mica with a hardness of 3 (see Fig. 567).

The limestones are stratified rocks composed of sedimentary or chemical deposits, of which the carbonate of lime forms the principal ingredient. When wholly crystalline and suitable for ornamental purposes it is called marble (Fig. 571). When it is composed largely of a double carbonate of lime and magnesia it is properly called dolomite. Some of the marbles also have this composition. When the stone is composed very largely of small shell fragments it is called oölitic limestone, Fig. 569 (from its resemblance to the roe of a fish). Onyx is a kind of crystalline limestone which has been formed wholly by chemical deposition, while stalactite and stalagmite formations are also limestones; but they should never be confounded with onyx, however much they may resemble it when polished.

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* The material in this chapter on *Stone* has been partly drawn from Merrill's *Stones for Building and Decoration* (Wiley, New York).

† The following scale of hardness is commonly used for minerals:

1. Easily scratched with the thumb-nail, as talc.
2. Can be scratched by the thumb-nail, as gypsum.
3. Not readily scratched by the thumb-nail, but readily cut with a knife, as calcite (calcspar, or calcium carbonate).
4. Can be cut with a knife less easily than calcite, as fluorite (fluor-spar).
5. Can be cut with a knife with difficulty, as apatite.
6. Can be cut with a knife only on thin edges, as feldspar.
7. Cannot be cut with a knife and scratches glass, as quartz.

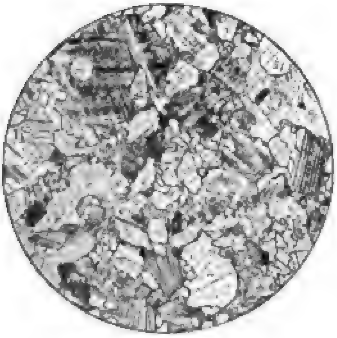

Fig. 567.—Biotite Granite, of Hallowell, Me.

Fig. 568.—Diabase from Weehawken, N. J.

Fig. 569.—Oölitic Limestone, Southern Indiana and Northern Kentucky.

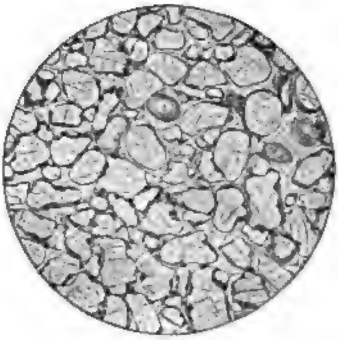

Fig. 570.—Reddish Potsdam Sandstone New York

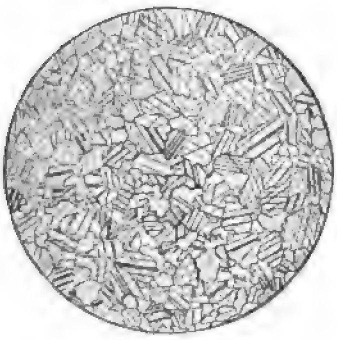

Fig. 571.—Crystallized Limestone or Marble from Vermont.

Fig. 572.—Brown Triassic Sandstone from Portland, Conn.

Microscopic Views of Building-stones. Magnified 20 Diameters. (From Merrill's *Stones for Building and Decoration*, 1891.)

Sandstones are fragmental rocks composed mostly of grains of silica (quartz) which have been cemented together by a deposition of silica, carbonate of lime, iron oxide, or clayey matter. If the cementing material be silica, as in Fig. 570, the rock, while extremely durable, is very hard and difficult to work. Iron oxide in the cementing material gives to the stone a reddish or brownish color, as shown in Fig. 572; here is also carbonate of lime and clayey matter, while the sand-grains are composed of both quartz and feldspar, this latter being indicated in the figure by the grains marked by parallel bands. This more porous and absorbent matrix is conducive to disintegration by water and frost, although such a stone is readily worked and has been very largely used in America. The most durable sand-stones (having the silica matrix) are so hard to work that other kinds of durable rock are generally preferred. When the sand-grains are so lightly attached that they will readily crumble they may be used as grindstones, as the sandstone of northern Ohio near Cleveland.

427. The Weathering of Building-stones.—This term includes the resistance of stones, when exposed to the weather, to all the disintegrating actions of heat and cold, water, frost, and chemical action, which combine in this climate to effect the rapid decomposition and destruction of most of the rocks, and of many of those which have been selected for building purposes. A stone building or monument should remain in good preservation for hundreds of years, but more commonly they begin to scale and crumble before they are twenty-five years old. The life of a rock may be many thousands of years in Egypt, or Italy, or Greece, when it would not last as many scores of years in the United States.

The chief disintegrating agent with the relatively impervious rocks is probably the variation of temperature, thus breaking the bond by continual expansions and contractions, while with the more porous and absorbent it is probably the freezing of the absorbed water. However, these two causes usually combine in this climate.

By far the best, and perhaps the only infallible, test of the weathering qualities of any given rock is the examination of a ledge of it which has been long exposed, or of an old building, slab, or monument of the stone from the same quarry and ledge. Sedimentary rocks, such as the limestones, may differ radically in consecutive ledges, so that here the particular course, or ledge, must be identified. As this test cannot be applied to a new quarry without an exposed face,* and because this is by far the most important quality of any building-stone, attempts have long been made to formulate artificial tests of this quality, but without any very marked success. A single illustration of actual weathering for many years is worth more than

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* North of the Ohio and Missouri rivers, where the face of the country has been scoured by glacial action these rock-exposures are common. Where the glacial erosion did not occur no sound rock-exposures should be expected. In the glaciated region the scratches and grooves of the glaciers are still plainly visible in many places.

all the artificial tests which can be applied. Nevertheless, when these cannot be had, something may be done in the way of determining relative resistance to frost, as described below.

428. Freezing Tests.—Specimens of the stone may be saturated with water and frozen and thawed say twenty-five times, after which the loss in dry weight and the loss in crushing strength may be determined. This is the method pursued by Prof. Bauschinger. The loss in weight by this method is so insignificant (see Fig. 573) as to furnish a very small base from which to estimate the relative weathering qualities of the stone. At best it is but a comparative test, the results of tests on various kinds of stone being compared with each other. The test for strength after freezing is also quite as unsatisfactory, since here comparison must be made between different specimens of the same stone, and a safe conclusion would have to rest upon average results of a large number of tests, and the results would still be only relative.

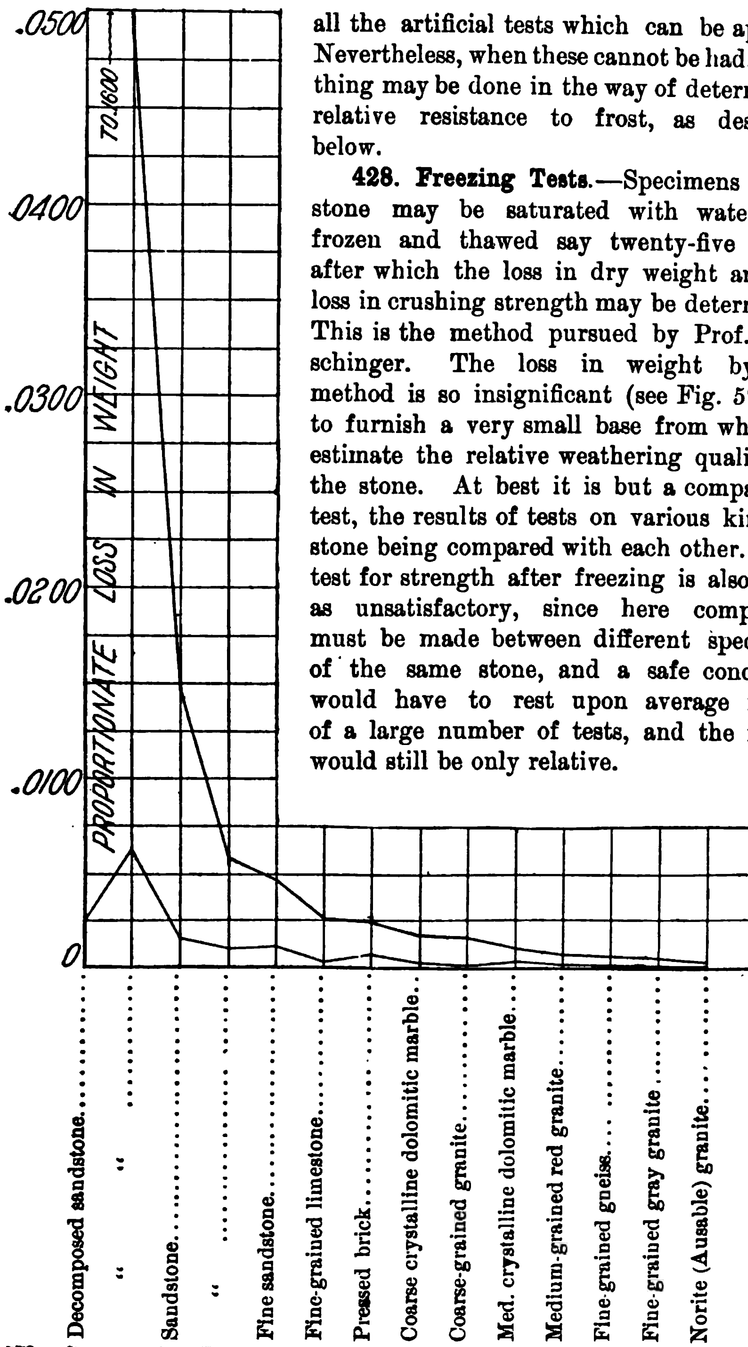

FIG. 573.—Comparative Tests of Building-stones by Freezings and by the Sulphate-of-soda Test. (*Trans. Am. Soc. C. E.*, vol. XXXIII. p. 242.)

429. The Sulphate-of-soda Test (Brard's process*) consists in immersing the specimens for a short time in a boiling solution of the sulphate of soda, in the form of a decahydrate, commonly known as *Glauber's salt* (Na_2SO_4, $10H_2O$), and then suspending them in the air for a day in order that the absorbed salt may crystallize. This process is repeated daily for a week or more, and then the dried salt is dissolved out by soaking in water, frequently renewed, for a week or more.

In making this test it is important to have a solution of the decahydrate which is saturated while cold or at a temperature below 80° F. The percentage required to make a saturated solution increases from 22 per cent at 32° F. to 120 per cent at 90° F. At about 100° F. the salt melts in its own combined water, and changes to the anhydrous form (Na_2SO_4), and at higher temperatures the anhydrous salt dissolves in water in diminishing proportions, reaching 126 per cent at 212° F.†. In cooling and drying, the decahydrate is again formed, and this crystallizes as the solution dries down below the saturation-point.

As the disintegrating action of this test is manifested wholly on the surface, so far as the loss of weight is concerned (the specimens being washed after each drying), it is important that the specimens tested should have the same superficial area per unit of weight. This means that if the specimens are of the same shape they must also be of the same size. A better method of representing the results would be to give the loss of weight per square inch of surface, rather than the percentage of loss in weight as is commonly given. Thus in the results plotted in Fig. 573 the specimens weighed from 23 to 94 grams, with no information given as to their shape or dimensions, while the results are taken out as percentages of loss in weight. Evidently the smaller specimens were at a great relative disadvantage, as their superficial areas would be much greater per unit of weight.

If the granites and the decomposed sandstones be omitted from the list of stones given in Fig. 573, the average loss of weight on the remainder by the sulphate-of-soda test is about six times that by the freezing test.‡

The specimens should either be heated before immersion in the boiling liquid, or they should be immersed before the liquid has come to the boiling temperature. The time of immersion need not be over 30 minutes, after which the specimens should be freely suspended in the open air for 24 hours. They are then sprayed from a wash-bottle and again immersed and boiled, this process being repeated for any desired number of times, generally from 7 to 10. The specimens should be small, about one-inch cubes being a suit-

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* *An. de Chem. et de Phys.*, vol. XXXVIII. p. 160, 1828, afterwards modified by d'Héricart and de Thury. See Trans. Am. Soc. C. E., vol. XXXIII. p. 246.

† See Fig. 21, p. 133, of Newth's *Inorganic Chemistry.*

‡ This agrees with comparative results obtained by Mr. E. Gerber, *Trans. Am. Soc. C.E.*, vol. XXXIII. p. 253.

able size, as the weighings have to be done with great care on delicate balances to secure reliable results.

The specimens must be carefully dried, for 24 hours, at a temperature above boiling, before the first weighing and after the long soaking in fresh water subsequent to the tests. Any neglect of this precaution may entirely vitiate the results because of the relatively large amount of water absorbed by the stone.

While this test may do injustice to some stones (possibly because of some chemical action upon them) which may resist weather exposure better than might be determined from these results, yet it seems to be the best artificial means of indicating a tendency to disintegrate on exposure which has yet been found. But, as compared to examples of actual exposure for long periods of time, it should be regarded as of no weight whatever.

430. Chemical Tests of stone are of little value in themselves, but taken in connecion with tests of strength, absorption, frost, and especially in connection with microscopical examinations, they may be of considerable value. In fact, as a general rule no chemical test need be made unless it is required to explain the microscopic structure, already examined as described in the following article.

431. The Microscopic Test consists in the preparation of very thin slices or sections of the stone and examining these under a microscope. A thin chip of the stone is first ground with emery to a plane on one side and polished. This side is then glued to a piece of glass with Canada balsam, and the other side ground and polished in a similar manner. The section is made so thin that it is quite translucent and sufficiently transparent to read a printed text through it. It is then mounted regularly upon a microscope-slide, and covered in the usual manner.

The examination under the microscope (magnifying about 25 diameters) consists in—

1. The observation of the structural arrangement.
2. The identification of its constitutent parts (by an experienced lithologist).
3. A study of the character of the cementing bond.

Thus, Fig. 567 * reveals a granite of very complex structure, wholly crystalline (though the crystals are greatly distorted), composed of quartz, several kinds of feldspar, and two kinds of mica. There may also be identified a crystallized phosphate of lime (apatite), in small needles, grains of iron ore, and occasionally small garnets.

Fig. 568 shows a wholly crystalline rock, with the crystals more perfect and much more firmly interlocked than those in Fig. 567. There are no openings found, and the component parts are feldspar, augite, and magnetite (or iron ore, which is the black or opaque portion). All these constituents

* The descriptions here given of Figs. 567 to 572 are condensed from Merrill's *Stones for Building and Decoration*, pp 34–37.

are both hard and tough, and they are so thoroughly bonded that this rock resists impact better than granite.*

In Fig. 569 is shown an oölitic limestone from Kentucky. It is composed of amorphous accretions (the dark portions) about fragmentary nuclei, all being cemented together by a crystalline formation of pure calcite (carbonate of lime, $CaCO_3$, hardness 3).†

It has been found to weather perfectly, works easily when first quarried, hardens on drying out, and is readily carved in most intricate patterns.

In Fig. 570 we have a silicious sandstone (the Potsdam, from the town of Potsdam, N. Y.), that is, a sandstone in which the pure sand (silica) grains are also cemented together by a deposit of pure quartz (silica) so that the whole is nearly a pure quartz or a quartzite. This is impervious to water and is unaffected by all other atmospheric agencies or gases, and hence weathers perfectly. It is, of course, very hard to work.

Fig. 571 is a good illustration of a wholly crystalline limestone or marble. In this transformed condition it is without cleavage planes, all traces of its original sedimentary formation having disappeared by metamorphism. The individual crystals have cleavage planes, shown by the dark stripes, but these do not follow any law as to direction.

Fig. 572 is of a brown (Triassic) sandstone from Connecticut, such as has been used so largely for residence fronts through the eastern and middle states. The original grains consist of quartz, the clear parts, and feldspar, the clouded and banded portions. These are cemented together by carbonate of lime, clayey matter, and iron oxide, this last being the black (opaque) portions. This kind of sandstone does not weather very well, because of its poor (clayey) cementing material.

The microscopic test, when made by an expert lithologist, will reveal more accurately the character and physical composition of a rock than any other single test. After such an examination the question of further study upon the specimen can be decided more intelligently.

432. The Absorption Test has for its object to determine the porosity or the percentage of water (by weight or by volume, or both) which may be absorbed by a dry specimen of the rock. Since some moisture may be assumed to be in any rock under normal conditions, it is necessary to first dry it for many hours at a temperature above 212° F. It is then weighed

* This trap-rock, diabase, is an igneous formation, forced up through the Triassic sandstones from Nova Scotia to North Carolina, forming great dikes, prominent examples of which are found in the Palisades of the Hudson, and Mount Holyoke and Mount Tom in western Massachusetts. It is much used for paving, foundation-walls, and also for monuments, when it is known as "black granite."

† This rock is sedimentary but massive, without seams or cleavage planes, and is found widely spread over southern Indiana and northern Kentucky. It has now come to be regarded as the leading building-stone of this country. It is commonly known as Bedford stone, from the town of Bedford, Ind., where it was first quarried for the general market.

and placed in clear water for one or more days, depending on the size of the specimen and the degree of accuracy desired. Evidently the air imprisoned in the rock largely prevents the entrance of the water. An exhaustion of this air under a receiver may considerably increase the percentage of absorption. On removing the specimen from the bath it is wiped dry and weighed again. The difference in weight divided by the dry weight gives the percentage by weight of moisture absorbed. To find the percentage by volume multiply the percentage by weight by the specific gravity of the stone. This is much the more significant relation. A series of repeated weighings while in the bath will indicate when it has become fully saturated. Table XXXVII gives results on a great variety of American building-stones.

433. The Specific-gravity Test.—The specific gravity, or weight per cubic foot, of any building-stone is important and should always be found. For hydraulic construction the heavier stones are far more valuable than the lighter ones, since the stability of the structure depends on the excess of the weight of the stone over that of the same volume of water. Thus, take two stones whose specific gravities are respectively 2.8 and 2.0 (175 and 125 lbs. per cubic foot); their actual weights are as 1.4 to 1, but their relative weights in water are as 1.8 to 1. That is, the heavier stone has 80 per cent more value than the lighter one in a retaining wall or dam. The solidity of a stone, or its freedom from small and microscopic cavities, may also be argued from its weight.

The most ready and accurate means of obtaining the specific gravity is by weighing it out of water and then under water; but as there will be some absorption when placed in the water, the specimen should first be weighed dry, then after soaking in water, as for the absorption test, and finally weighed under water. Then the specific gravity can be computed either for the wet or for the dry stone as may be desired.

Thus, let W_d = weight dry;
W_w = " wet;
W_u = " under water.

Then we have

$$\textit{Specific gravity dry} = \frac{W_d}{W_w - W_u}. \qquad (1)$$

$$\textit{Specific gravity wet} = \frac{W_w}{W_w - W_u}. \qquad (2)$$

(See Table XXXVIII for the specific gravities of the leading varieties of American building-stones.)

434. The Compressive Strength.—The laws governing the compressive strength of brittle materials, as stone, were given in Chapter III. It was there shown that in order to develop a normal failure in compression it is necessary to use specimens having a length in the direction of the stress at

least 1.5 times the least lateral dimension. As a matter of fact, however, cubical forms have been almost exclusively used for this purpose, so that in order to obtain results which shall be comparable with those which have come to be regarded as standard, it is necessary to continue this custom. The relative strength of other forms of prisms may be found from Fig. 17, p. 32.

In Chapter XVI the proper methods to be employed in making compression tests were given, so that these need not be entered upon here. Since all stones which would be commonly regarded as suitable for building purposes are far stronger than is actually required, it follows that the compression test is really of very little consequence, and yet this is in many instances the only test which is asked for on a new building-stone. From Table XL it appears that even the lightest sandstones have a

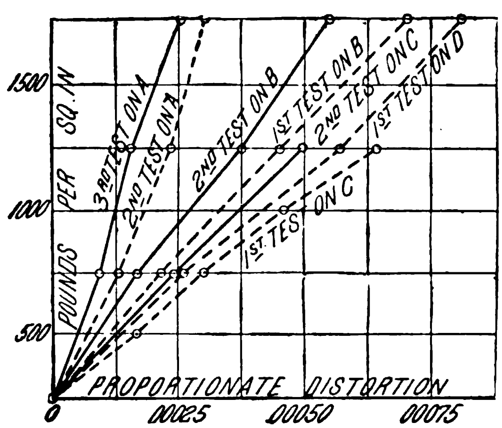

FIG. 574.—Compression Tests on Four Kinds of English Limestone. (*Inst. Civ. Engrs.*, vol. CVII.)

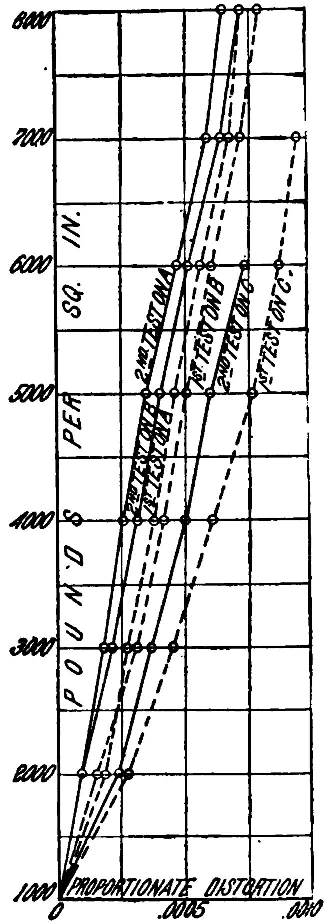

FIG. 575.—Repeated Compression Tests on Three Kinds of Scotch Granite. (*Inst. Civ. Engrs*, vol. CVII.)

compressive strength of from 4000 to 6000 lbs. per square inch when tested in cubical forms. From Fig. 17, p. 32, it appears that the crushing strength of stone pillars and columns would be at least 0.8 of these amounts. But the greatest load ever placed on stone or brick masonry is commonly not over about 10 tons per square foot or 140 lbs. per square inch.* Any stone,

* This corresponds to the weight of a prism 267 feet high built of masonry weighing 150 lbs. per cubic foot.

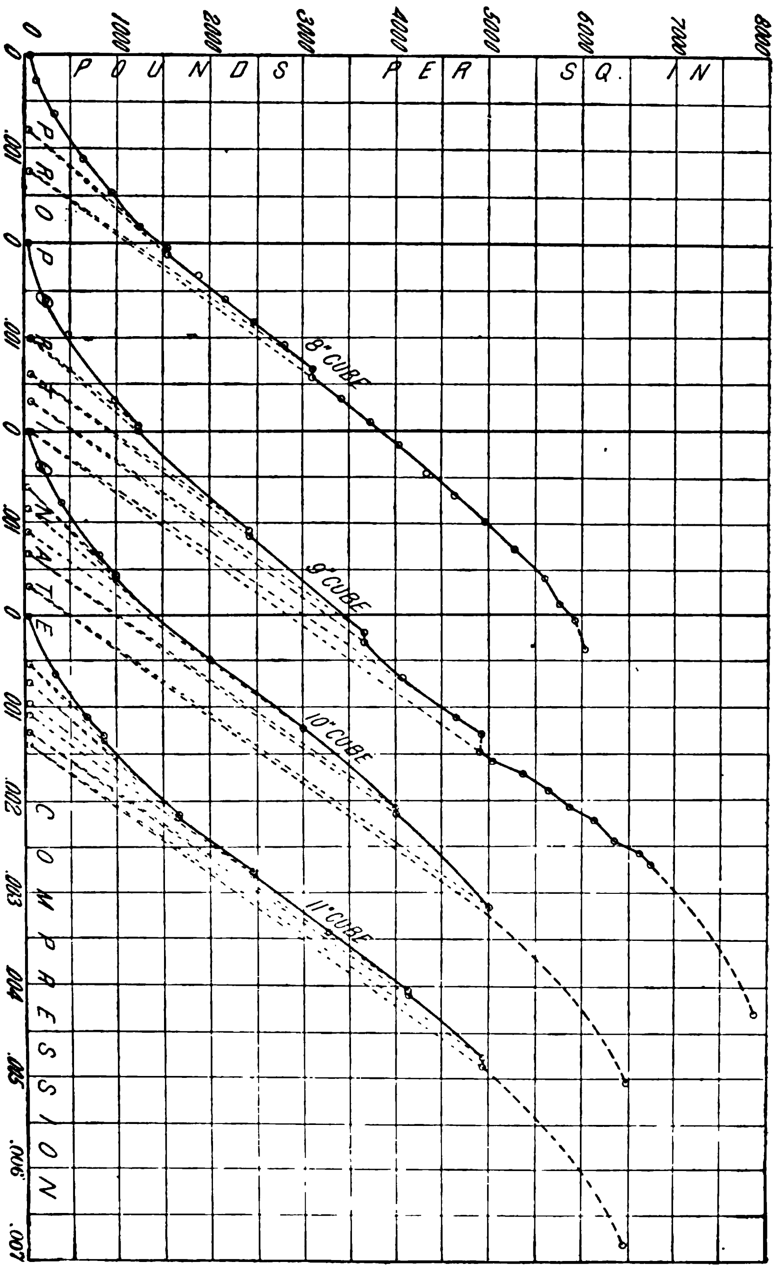

FIG. 576.—Compression Tests on Free (Sand) Stone Cubes. (Gillmore.)

TABLE XL.—PHYSICAL PROPERTIES OF BUILDING-STONES.

Condensed from Merrill's *Stones for Building and Decoration.*

Kind of Stone.	Locality.	Position.	Strength per Square Inch.	Specific Gravity.	Weight per Cubic Foot.	Percentage of Absorption by Weight.	Number of Specimens averaged.
			lbs.		lbs.		
1. Granite	Grape Creek, Brownsville, Lawson, Platte Cañon, Cotopaxi, Monarch, Gunnison—*Colo.*	Bed Edge	15,531 18,536	2.68	187.3	1.1	8
2. Granite	New London, Millstone Point, Mystic River, Stony Creek—*Conn.* Vinalhaven, Fox Island, Dyer's Island, City Point, Dix Island, Jonesboro, Sprucehead, Hewitt's Island, Hurricane Island—*Maine.* Huron Island—*Mich.*	Bed	16,200	2.65	166.0	0.4	20
3. Granite	East Saint Cloud, Saint Cloud, Watab, Sauk Rapids, Beaver Bay—*Minn.*	Bed Edge	24,464 24,464	2.65	165.8	0.5	7
4. Granite	Cape Ann, Rockport, Quincy—*Mass.*	Bed	16,079	2.67	167.0	0.7	4
5. Granite	Fall River, Monson — *Mass.* Keene — *N. H.* Tarrytown, Morrisania, Staten Island, North River, Madison Avenue, Chaumont Bay—*N. Y.* Westerly — *R. I.* Richmond —*Va.*	Bed	15,570	2.69	168.0	0.4	14
6. Granite	New Haven — *Conn.* Duluth, Taylor's Falls, Beaver Bay—*Minn.* Jersey City Heights, Pompton—*N. J.* Goose Creek (Loudoun County)—*Va.*	Bed Edge	21,272 20,740	2.82	176.2	0.3	6
7. Limestone (oölitic)	Putnamville, Greensburgh, Saint Paul, Harrison County, Mount Vernon, Bloomington—*Ind.*	Bed	14,054	....	156.2	1.4	6
8. Limestone	Spencer, Ellettsville, Bedford, Salem—*Ind.*	Bed	9,297	....	145.9	3.6	8
9. Limestone	Bardstown—*Ky.*	Bed Edge	16,250 15 000	2.67	166.9	1.2	1
10. Limestone	Lee—*Mass.*	Bed Edge	22,328 21,728	....	....	...	3
11. Limestone	Frontenac, Stillwater, Winona, Red Wing, Kasota, Mantorville—*Minn.*	Bed Edge	16,320 16,648	2.52	157.3	3.1	7
12. Limestone	Glens Falls, Lake Champlain, Canajoharie, Kingston, Garrison's Station, Williamsville—*N. Y*	Bed Edge	16,971 15,533	2.58	168.1	....	6

PHYSICAL PROPERTIES OF BUILDING-STONES—*continued.*

Kind of Stone.	Locality.	Position.	Strength per Square Inch.	Specific Gravity.	Weight per Cubic Foot.	Percentage of Absorption by Weight.	Number of Specimens averaged.
			lbs.		lbs.		
13. Limestone (marble)	Montgomery County—*Pa.*	Bed Edge	18,112 11,055	....	....	....	4
14. Limestone (marble)	Dorset—*Vermont.*	Bed Edge	10,506 8,670	2.64 2.68	164.7 167.8	....	2 1
15. Limestone (marble)	Italy.	Bed	12,156	2.69	168.2	....	1
16. Sandstone	Buckhorn (Larimer Co.), Trinidad (Las Animas Co.), Manitou (El Paso Co.), Ralston, Left Hand, Saint Vairus, Fort Collins (Larimer Co.), Stout (Larimer Co.)—*Colo.* Thistle —*Utah.*	Bed Edge	11,141 12,434	2.18	132.9	6.6	9
17. Sandstone	Coal Creek, Oak Creek (Fremont Co.), Gunnison (Gunnison Co.), Manitou (El Paso Co.), La Porte (Larimer Co.), Brandford (Fremont Co.)—*Colo.*	Bed Edge	5,481 4,941	2.12	133.0	13.8	9
18. Sandstone	Middletown, Portland — *Conn.* East Long Meadow — *Mass.* Marquette—*Mich.*	Bed	6,639	2.27	142.2	3.5	3
19. Sandstone	Hinckley, Fort Snelling—*Minn.*	Bed Edge	16,625 18,750	2.38	139.0	6.0	2
20. Sandstone	Dresbach, Jordan, Fond du Lac, Dakota—*Minn.*	Bed Edge	5,789 4,102	1.99	124.4	9.9	6
21. Sandstone	Taylors' Falls, Kasota, Frontenac—*Minn.*	Bed Edge	7,483 9,725	2.42	142.4	5.9	3
22. Sandstone	Haverstraw, Hudson River, Albion—*N. Y.*	Bed Edge	8,925 7,687	2.78	142.2	2.6	2
23. Sandstone	Medina—*N. Y.*	Bed Edge	17,500 14,812	2.42 2.39	150.8 149.3	1.6 2.0	2 1
24. Sandstone	Vermillion—*Ohio.*	Bed Edge	7,840 6,875	2.16	135.0	5.2	5 1
25. Sandstone	Seneca—*Ohio.*	Bed Edge	9,687 10,500	2.39	149.3	3.1	1
26. Sandstone	Cleveland—*Ohio.*	Bed Edge	6,800 7,910	2.24	140.0	2.8	1
27. Sandstone	Marblehead—*Ohio.*	Bed Edge	7,937 6,850	2.31	144.4	5.2	1
28. Sandstone	North Amherst—*Ohio.*	Bed Edge	6,212 5,450	2.16	133.7 135.8	5.2	2 1
29. Sandstone	Berea—*Ohio.*	Bed	9,236	2.13	133.0	5.5	2

therefore, having a crushing strength of 3000 lbs. per square inch is quite strong enough for all ordinary building purposes, the strength of masonry being measured by the strength of the mortar used. While there is no objection to greater strength, and while strength may be some evidence of weathering resistance, yet it cannot be said that one stone is better than another for building purposes simply because its crushing strength is 20,000 lbs. per square inch, whereas the strength of the other is only 5000 lbs. per square inch. In the opinion of the author this difference of strength, taken alone, has no significance and should be given no weight.

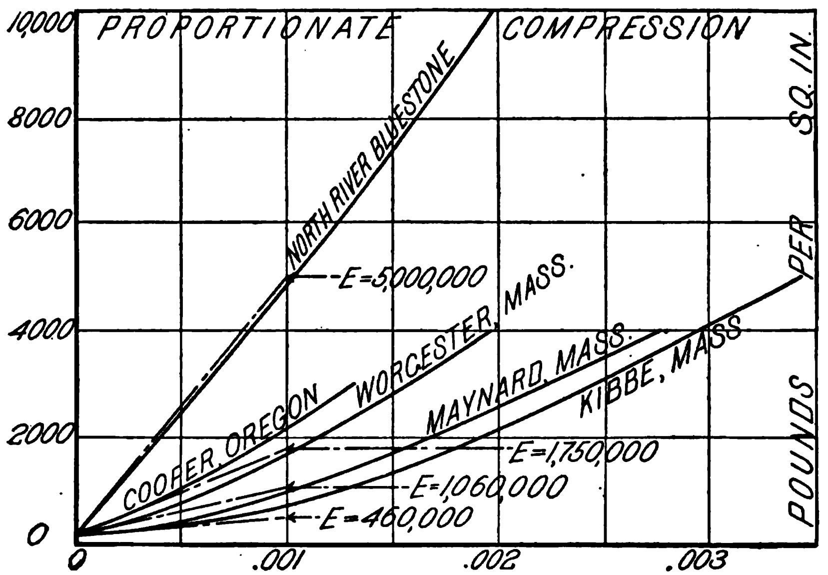

FIG. 577.—Elastic Properties of Various Stones under Compressive Stress. (*Wat. Ars. Rep.* 1894.)

435. The Elastic Properties and Crushing Strength of Building-stones.—Limestones and granites are nearly perfectly elastic for all working loads, while sandstone takes permanent sets for the smallest loads. These qualities are well illustrated in Fig. 574 for limestone, Fig. 575 for granite, and Fig. 576 for sandstone. These figures show that some permanent set accompanies all loads even on the limestones and granites, but these are extremely small as compared to those on the sandstone. Similar results appear also on Figs. 577 to 583.

In all these stress-diagrams the modulus of elasticity may be taken off by extending a tangent to the curves at the origin till it intersects the vertical line marking a deformation of 0.001 of the length. The corresponding load in pounds per square inch, taken from the stress argument, when multiplied

by 1000 gives the modulus of elasticity in compression. Bauschinger has shown that this is practically the same as that obtained from bending tests, and hence it may be used for computing the deflection of stone beams. The moduli of elasticity given in Table XLI are those found in cross-bending on the first loading. Bauschinger gives them also in every case for tension and for compression; but these are not reproduced here, as they are practically the same as in cross-bending. By comparing the results in this table three seems to be no general or fixed relation between the various kinds of strength of stone. As these tests were made with the greatest care and precision, this table must be accepted as conclusive on this point.

TABLE XLI.—PROPERTIES OF THE BUILDING-STONES OF BAVARIA.

(Bauschinger's *Communications*, vol. x, 1884.)

Kind of Stone.	Specific Gravity.	Weight per Cubic Foot.	Cross-bending.		Compressive Strength.			Tensile Strength.	Shearing Strength.	
			Modulus of Elasticity	Modulus of Rupture.	Perpendicular to Bed.	Parallel to Bed.	Parallel to Bed after 25 Freezings.		Perpendicular to Bed.	Parallel to Bed.
Granite	2.65	165.4	2,986,000	1,365	19,200	18,910	21,470	619	1,379	142
"	2.66	166	1,621,000	1,194	19,200	20,050	20,480	683	1,450	853
Triassic limestone	2.48	154.8	6,420,000	882	8,130	8,320	6,810	583	555	384
Jurassic limestone (marble)	2.23	139.2	......	...	11,110	7,410	12.290	448	739	540
" " "	2.08	129.8	4,906,000	462	4,664	8,760	3.313	213	498	299
Oölitic limestone	2.72	169.7	....	1,792	19,340	20,620	18,770	910	1,479	1,138
Tuffa stone	1.80	112.3	....	469	1,195	2,545	2.076	227	227	213
Variegated sandstone	2.06	128.5	426,600	469	7,420	6,010	6,730	107	569	355
" "	2.20	137.3	867,400	718	9,040	7,790	7,910	199	512	313
" "	2.28	142.3	1,340,000	1,109	12,980	13,410	11,520	576	910	540
" "	2.00	124.8	341,300	341	6,160	6.100	4,877	128	455	427
Carboniferous sandstone	2.20	137.3	910,000	488	7,636	8,390	5,986	341	640	284
" limestone	2.23	139.1	334,200	441	6,684	6,670	5,900	213	588	469
Slaty sandstone	1.82	113.6	512,000	249	3,071	2,247	2,161	98	370	242
" "	1.92	119.8	270,200	135	3,029	2,659	4,252	67	242	185
Green sandstone	2.15	134.2	583,000	156	4.707	4.308	4,038	94	341	327
Cretaceous sandstone	2.60	162.3	568,800	597	13,510	14,500	......	327	668	370
" "	2.73	170.4	2,687,000	967	28,860	17.490	....	512	995	768
Quartz conglomerate	2.29	142.9	1,763,000	654	5,546	4,408	3,270	242	....	....

In Table XLII we have given, in addition to the usual compressive strength, the modulus of elasticity in compression, the shearing strength, the ratio of lateral to longitudinal deformation from stress (Poisson's ratio, see Art. 5, p. 5), and the coefficient of expansion. This last property of stone the author has not found elsewhere, and as it is a very important one, the table has great value for this alone. The coefficient of expansion of iron and steel is about 0.0000065 (varying from 0.0000050 to 0.0000070), while that of stone and cement, as shown by this table, varies from about 0.0000020 to 0.0000060. Iron or steel embedded in stone masonry, therefore, would have a very small relative expansion and contraction from temperature. It will be noticed that Poisson's ratio varies from $\frac{1}{5}$ to $\frac{1}{3}$, so that the value $\frac{1}{4}$,

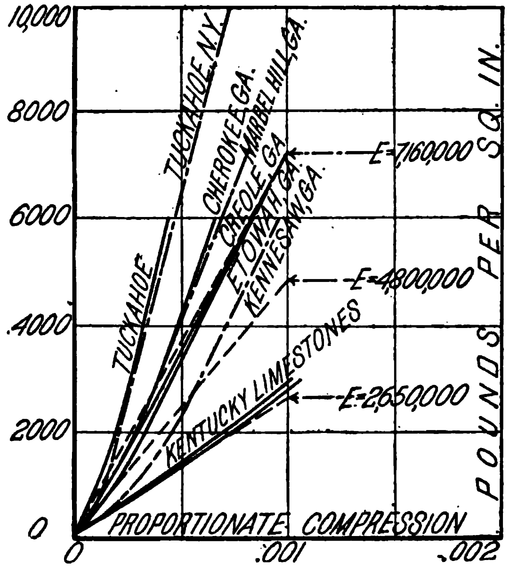

FIG. 578.—Elastic Properties of Various Stones under Compressive Stress. (*Wat. Ars. Rep.* 1894.)

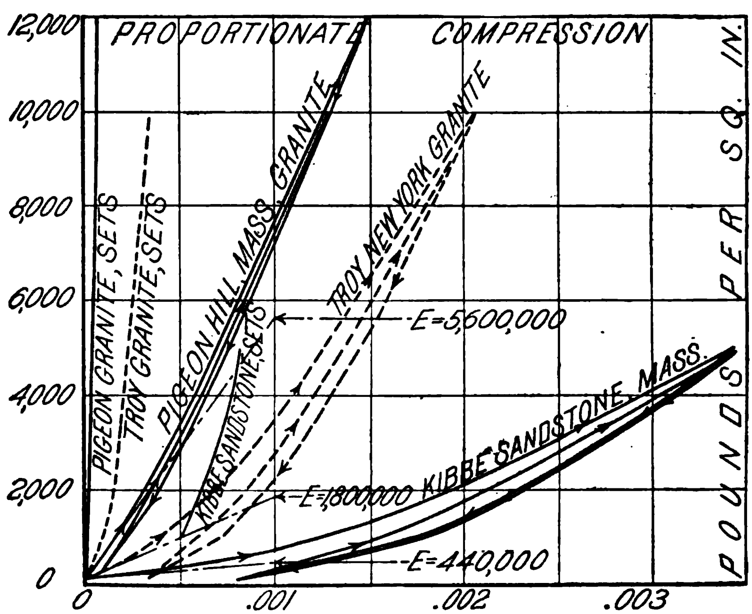

FIG. 579.—Elastic Properties of Various Stones under Compressive Stress. (*Wat. Ars. Rep.* 1894.)

which is commonly assumed for metals, would also serve very well for stone.

TABLE XLII.—TESTS OF AMERICAN BUILDING-STONE MADE AT THE WATERTOWN ARSENAL.

(Rep. 1894.)

Name of Stone.	Weight per Cubic Foot.	Compression Tests. Strength in Pounds per Square Inch.	Compression Tests. Modulus of Elasticity for Working Loads.	Ratio of Lateral Expansion to Longitudinal Compression.*	Shearing Strength.	Coefficient of Expansion in Water.
	Pounds				Pounds.	
Brandford granite (Conn.)...	162.0	15,707	8,333,300	0.250	1,833	.00000398
Milford granite (Mass.)......	162.5	23,775	6,663,000	0.172	2,554	.00000418
" " "	....	......	....	....	....	.00000415
Troy granite (N. H.)........	164.7	26,174	4,545,400	0.196	2,214	.00000337
Milford pink granite (Mass.).	161.9	18,988	5,128,000	....	1,825	....
Pigeon Hill granite (Mass.)..	161.5	19,670	6,666,700	....	1,550	....
Creole marble (Georgia). ...	170.0	13,466	6,896,500	0.345	1,369	....
Cherokee marble (Georgia)..	167.8	12,618	9,090,900	0.270	1,237	.00000441
Etowah marble (Georgia)...	169.8	14,052	7,843,100	0.278	1,411	....
Kennesaw marble (Georgia)..	168.1	9,562	7,547,100	0.256	1,242	....
Lee marble (Mass.)...... ...	....	......	....	....	....	.00000454
Marble Hill marble (Georgia)	168.6	11,505	9,090,900	0.294	1,332	.00000194
Tuckahoe marble (N. Y.)....	178.0	16,203	13,563,200	0.222	1,490	.00000441
Mt. Vernon limestone (Ky.)..	139.1	7,647	3,200,200	0.250	1,705	.00000464
Oolitic limestone (Ind.)......	...	......	....	....	....	.00000437
North River bluestone (N.Y.)	....	22,947	5,268,800	....	....	....
Monson slate (Maine)........	....	......	....	....	....	.00000519
Cooper sandstone (Oregon)..	159.8	15,163	2,816,900	0.091	1,831	.00000177
Sandstone, Cromwell (Conn.)	....	10,780	....	...	....	....
Maynard sandstone (Mass.)...	133.5	9,880	1,941,700	0.333	1,204	.00000567
Kibble sandstone (Mass). ...	133.4	10,363	1,834,900	0.300	1,150	.00000577
Worcester sandstone (Mass)..	136.6	9,762	2,439,000	0.227	1,242	.00000517
Potomac sandstone (Md.)....	....	......	....	....	....	.00000500
Olympia sandstone (Oregon).	....	12,665	....	...	....	.00000320
Chuckanut sandstone (Wash.)	....	11,389	....	....	1,352	....
Dyckerhoff Portland cement, neat....................	....	......	....	....	....	.00000578

* See Art. 5, p. 5.

436. Resistance to Abrasion.—When a stone is strong and tough enough to resist the chipping action of the iron horseshoes and wagon-tires upon it when used as a paving material, and when it weathers perfectly, its life is measured by its resistance to the abrading action of the traffic. Prof. Bauschinger very fully investigated this subject, and his results are recorded in volume XI of his *Communications* (1884). We here find some 900 tests of paving materials, most of which are summarized by averages in Table XLIII. His apparatus is shown in Fig. 585, which is modelled after a similar machine shown at the world's fair held in Paris in 1878. The cut as here shown is to a scale of one-half inch to the foot, so that the diameter of the revolving table was about five feet. Any given specimen was held to a fixed position

on the plate, two specimens being tested at one time. The specimens were all dressed to 4 inches (10 cm.) square, and they were weighted with 30 kilograms, or about 4 lbs., per square inch. Tests were made both with and

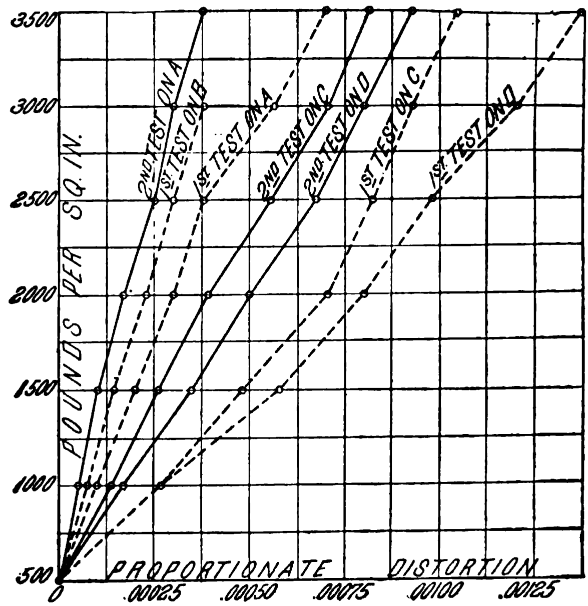

FIG. 580.—Compression Tests on Four Kinds of English Dolomite. (*Inst. Civ. Engrs.*, vol. CVII.)

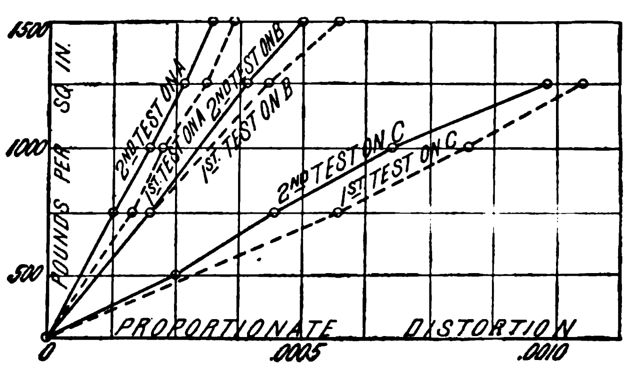

FIG. 581.—Compression Tests on Three Kinds of Oolitic Limestone. (*Inst. Civ. Engrs.*, vol. CVII.)

without the use of water, but mostly without, as shown in the table Fine emery (No. 3) was fed to the plate by hand at the rate of 20 grams for every 10 revolutions, the old emery being at the same time brushed off. Two attendants constantly kept the emery in the path of the specimen. The

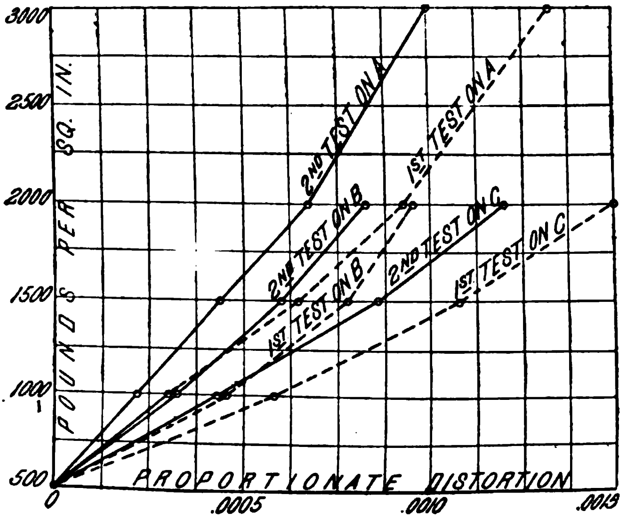

FIG. 582.—Compression Tests on Three Kinds of English Sandstone. (*Inst. Civ. Engrs.*, vol. CVII.)

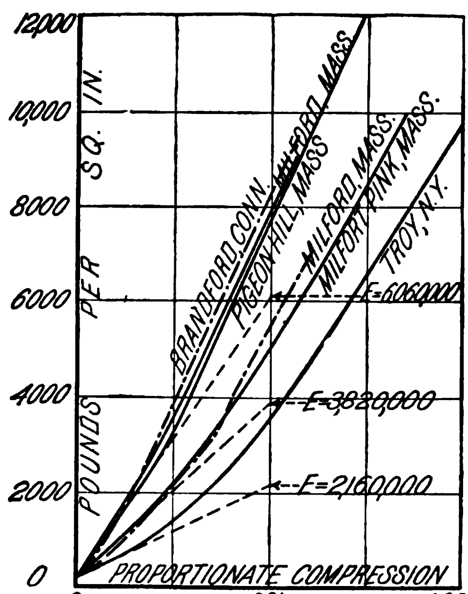

FIG. 583.—Elastic Properties of Various Stones under Compressive Stress. (*Wat. Ars. Rep.* 1894.)

speed was 20 revolutions per minute, and 200 revolutions completed a test, so that the test lasted but 10 min. The different specimens were set at different distances from the centre, so as to wear the cast-iron table evenly, and the results were all reduced to a standard radius (distance from the centre) of 49 cm. (19.5 in.). Elaborate preliminary studies were made to

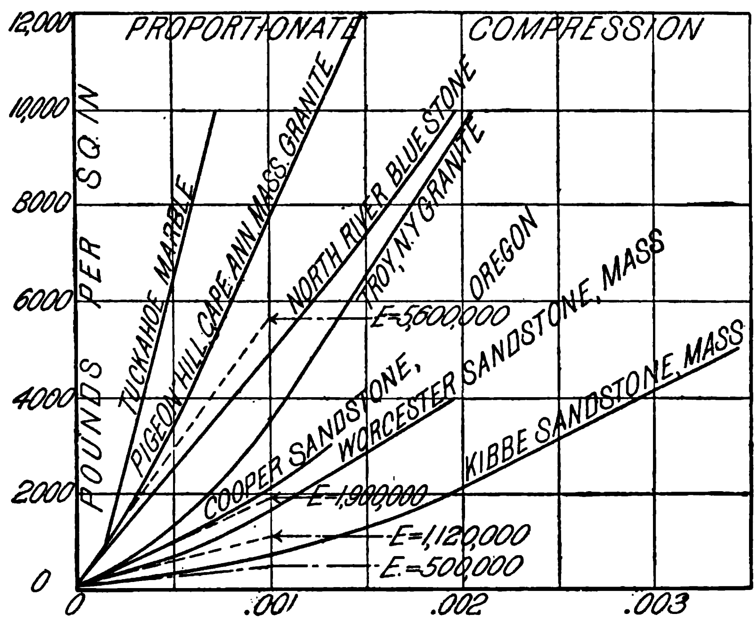

FIG. 584.—Elastic Properties of Various Stones under Compressive Stress. (*Wat. Ars. Rep.* 1894.)

determine the best rate of feeding emery, best size of grain, best weighting of specimen, and the law of wear as the distance out from the centre varied. Some of the results of these preliminary tests are shown in Figs. 586 and 587. The average results as recorded in Table XLIII indicate:

1. That the wet grinding was about twice as effective as the dry grinding, the exact average ratios being given in the last column of the table for each species of stone.*

2. There is no fixed relation between crushing strength and abrasive resistance.

3. The limestones wear about five times and the sandstones about four times as fast as the granites, porphyries, and basalts.

* These ratios have been taken out from the wet and dry tests on identical material, and therefore are not the ratios of the two general average results in the previous column.

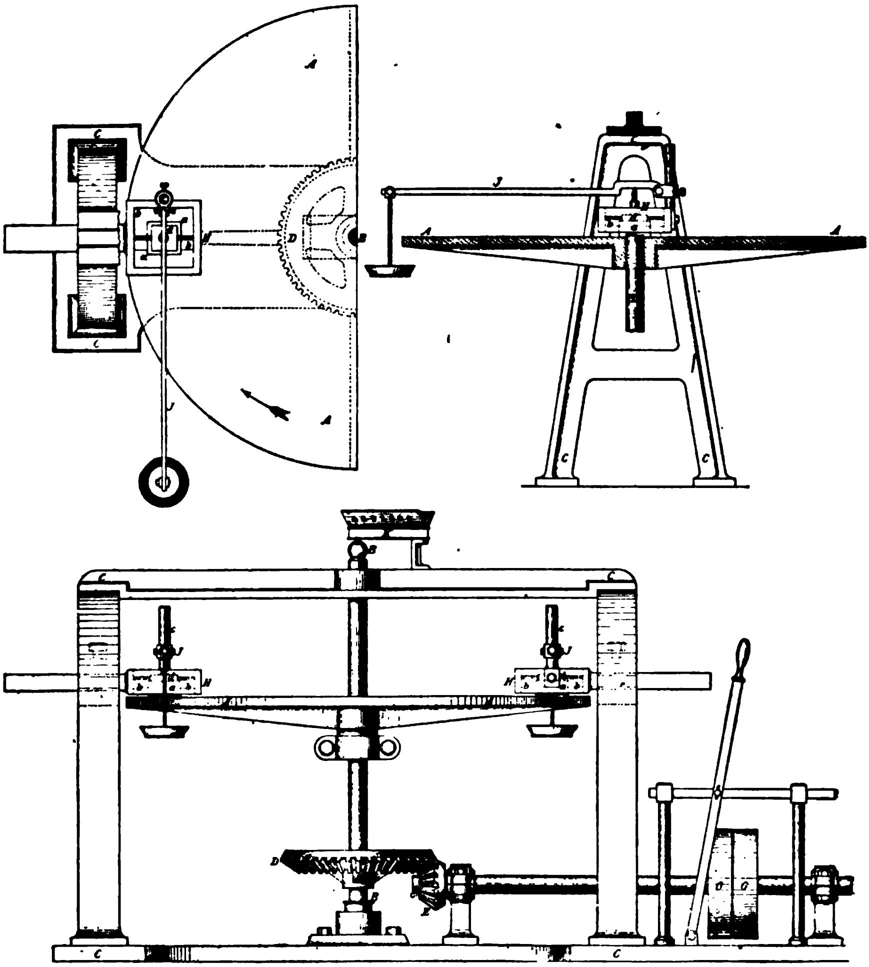

Fig. 585.—Bauschinger's Apparatus for Determining Resistance to Abrasion of Paving Material.

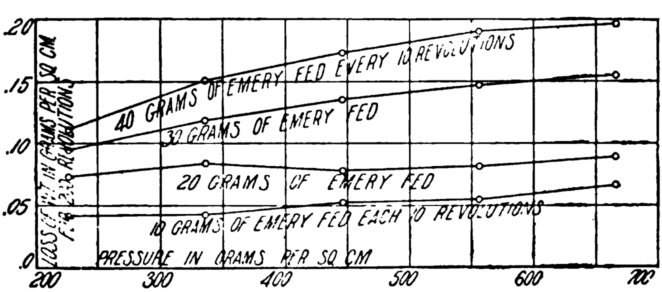

Fig. 586.—Showing the Relation between the Abrasion, Pressure, and Energy used. (Bauschinger.)

TABLE XLIII.—AVERAGE RESULTS OF BAUSCHINGER'S ABRASION TESTS OF PAVING MATERIAL.

(*Communications*, vol. XI, 1884.)

Four-inch cubes of the material were pressed on an iron plate with a weight of four pounds per square inch, and 20 grams of emery fed every 10 revolutions. Results obtained for 200 revolutions at a radius of 19.5 inches.

Kind of Material.	Average Specific Gravity.	Average Weight per Cubic Foot in Pounds.	Average Compressive Strength in Pounds per Square Inch.	Number of Results Averaged.	How ground: Dry or Wet.	Average Loss of Volume in Cubic Inches.	Ratio: Loss wet / Loss dry.
Granite	2.63	164	22,400	92 8	dry wet	0.24 0.46	1.72
Syenite	2.27	142	18,780	24 1	dry wet	0.28 0.82	1.90
Diorite	2.87	180	26,200	18 2	dry wet	0.27 0.68	1.90
Hornblende	2.82	176	21,900	2	dry	0.19	....
Porphyry	2.57	161	24,500	93 8	dry wet	0.20 0.24	1.72
Basalt	3.01	188	34,200	86 4	dry wet	0.19 0.47	2.31
Gneiss	2.61	163	23,000	4	dry	0.21	....
Quartz	2.63	165	17,500	9	wet	0.19	....
Clay-slate	2.72	170	26,000	8 2	dry wet	0.16 0.35	2.76
Breccia	2.61	163	22,600	10	dry	0.20	....
Limestone	2.87	180	20,500	163 32	dry wet	1.10 1.41	1.60
Sandstone	2.48	155	17,600	44 38	dry wet	0.81 0.64	2.25
Brick and tile	2.98	187	....	105 34	dry wet	0.38 0.75	2.50
Artificial stone made with Portland cement	2.36	148	....	20 4	dry wet	0.51 1.82	3.20
Asphalt paving	2.33	146	....	2 2	dry wet	0.61 1.62	2.68

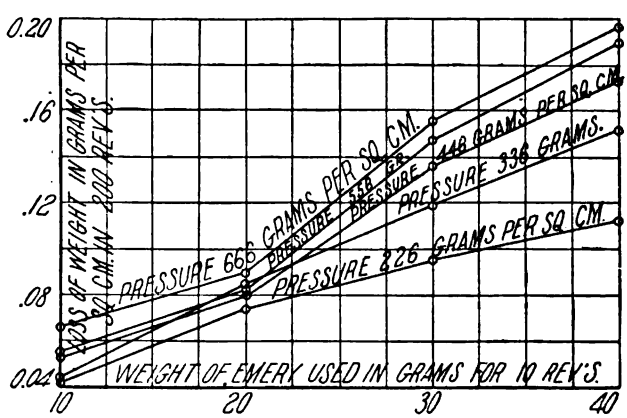

FIG. 587.—Showing the Relation between the Abrasion, the Emery used, and the Pressure exerted. (Bauschinger.)

4. The clay-slate shows the best results in abrasion, but only a few specimens were tested.

5. The brick and tile wear about twice as fast and the cement compositions about three times as fast as the primitive rocks.

6. The resistance of asphalt paving to abrasion falls between the cement mixtures and sandstone.

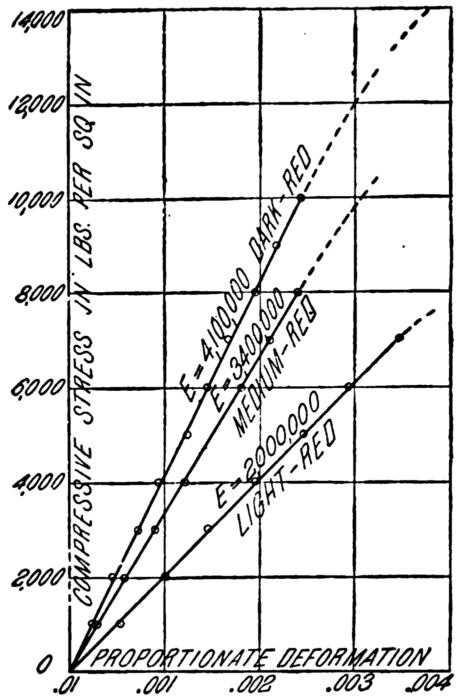

FIG. 588.

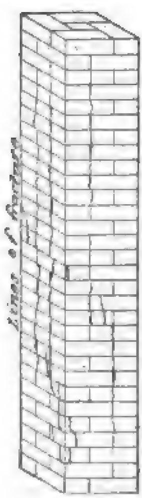

FIG. 589.

FIG. 588.—Elastic Properties of Common Brick used in Pier Tests. The average crushing strength of these three grades of brick, crushed endwise, was 14,000, 10,500, and 7500 lbs. per square inch respectively. (*Wat. Ars. Rep.* 1885, p. 1138.)

FIG. 589.—Showing Method of Failure of Brick Piers. (*Wat. Ars. Rep.* 1883.)

BRICK.

437. The Strength and Elastic Properties of single brick are of relatively small importance unless the mortar bond has nearly as great strength. As this is never the case except a rich Portland-cement mortar be used, it follows that in ordinary brick masonry the strength and rigidity of the brick used is of small importance provided any reasonably firm brick be employed. Thus in Fig. 588 we have three stress-diagrams of compression tests of single brick, showing moduli of elasticity from 2,000,000 to 4,000,000, and an

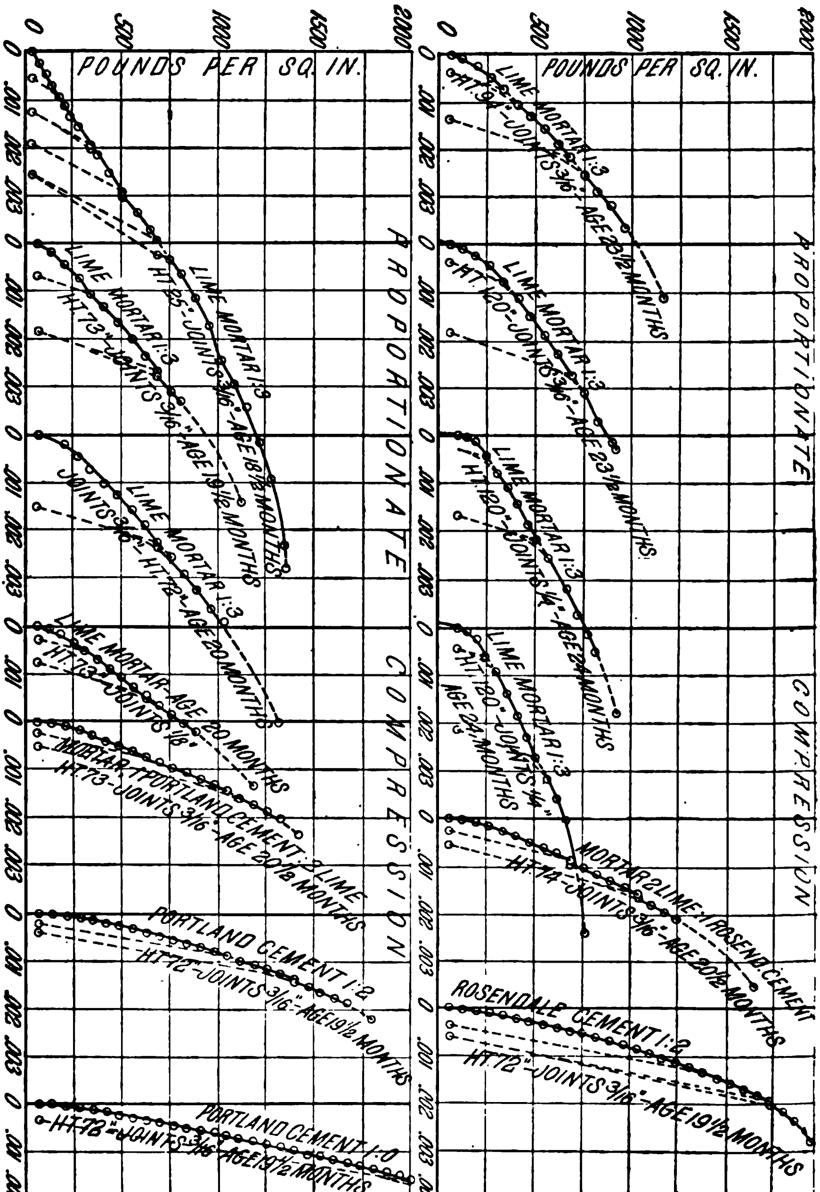

FIG. 590.—Strength of 12-in. Solid Bay State Brick Piers with Various Kinds of Mortar. (Wat. Ars. Rep. 1883.)

ultimate crushing strength from 8,000 to 14,000 lbs. per square inch. In Fig. 590 are shown the stress-diagrams of tests on columns from 6 to 10 feet high built from the strongest of the brick tested in Fig. 588, but with

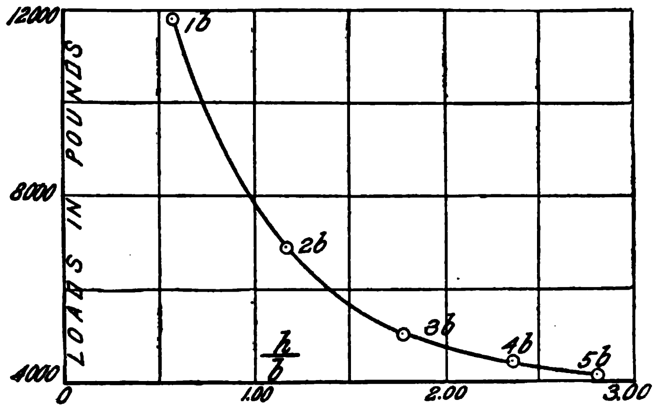

FIG. 591.—Strength of Columns of Single Hard-burned Eastern Face-brick laid flatwise one upon another with Plaster-of-paris Joints. Each result the mean of two tests. (*Rep. Wat. Ars.* 1894, p. 440.)

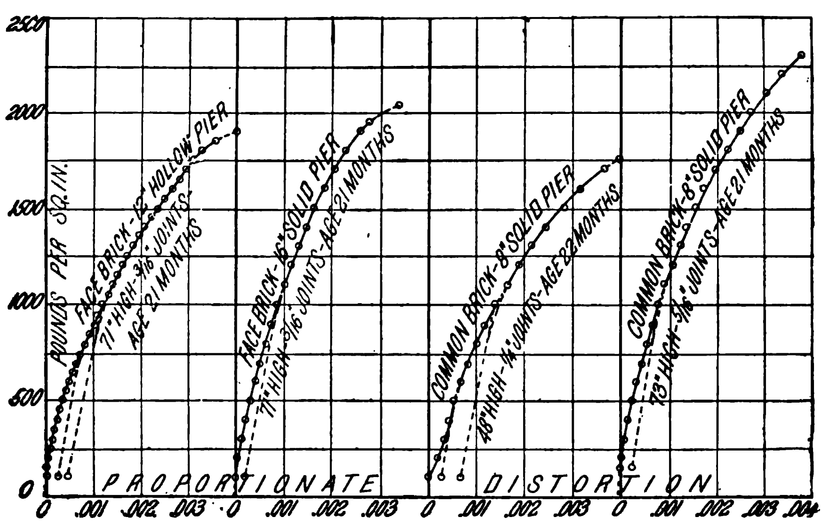

FIG. 592.—Strength of Brick Piers with Rosendale-cement-mortar Joints, 1 C. : 2 S. (*Wat. Ars. Rep.* 1883.)

various kinds of mortar. When lime-mortar (1 lime to 3 sand) was used at ages from 18 months to 2 years, the modulus of elasticity for the column as a whole varied for the first loads from 250,000 to 750,000, and the ultimate strength from 750 to 1300 lbs. per square inch. The method of failure

of all these columns is fairly indicated in Fig. 589. They always split longitudinally and spread apart, thus showing that the tensile strength of the brick is really a very important quality.

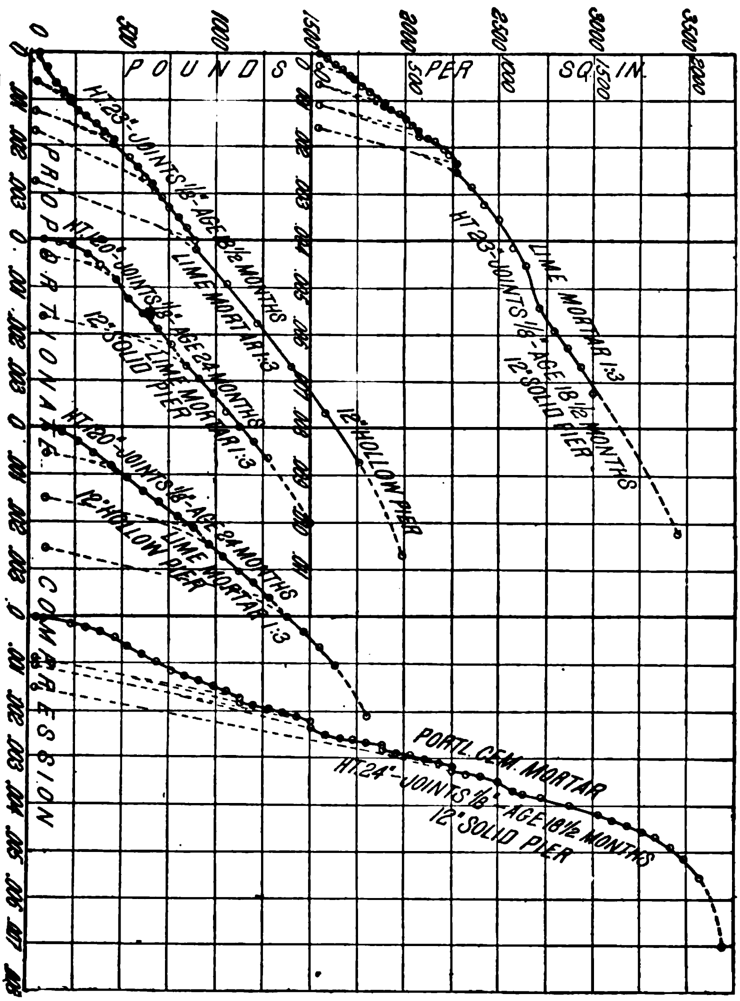

Fig. 598.—Strength of Face-brick Piers with Various Mortars. (Wat. Ars. Rep. 1883.)

When Rosendale (natural) cement mortar (1 C. : 2 S.) was used the modulus of elasticity was raised to about 2,000,000, and the ultimate strength to 2000 lbs. per square inch.

When Portland-cement mortar (1 C. : 2 S.) was used the modulus of elasticity of the column was raised 3,000,000, and the strength to 2500 lbs. per square inch.

The effect of adding one part of Rosendale or of Portland cement to two parts of lime and three parts of sand is shown by two diagrams on Fig. 590.

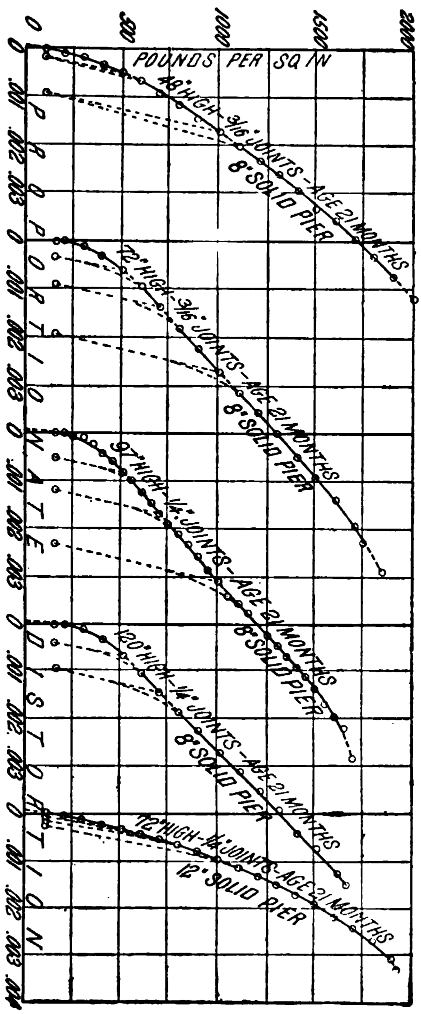

Fig. 594.—Fack-brick Piers with Rosendale-cement-mortar Joints. 1 C. : 2 S. (Wat. A'rs. Rep. 1888.)

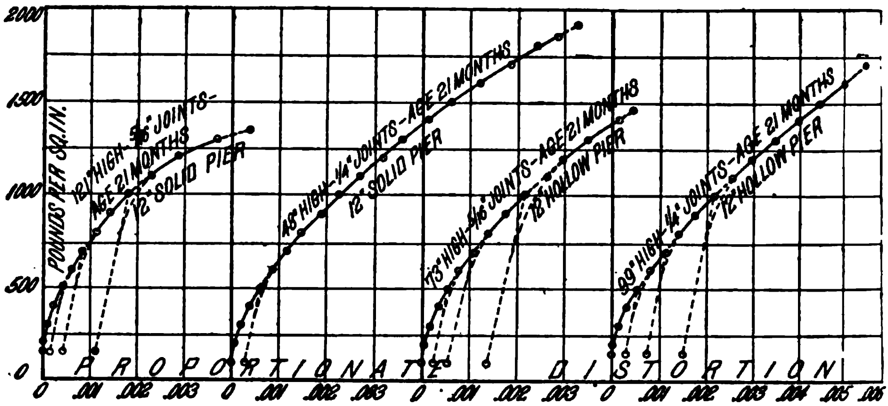

FIG. 595.—Strength of Common-brick Piers with Rosendale-cement-mortar Joints, 1 C. : 2 S. (*Wat. Ars. Rep.* 1883.)

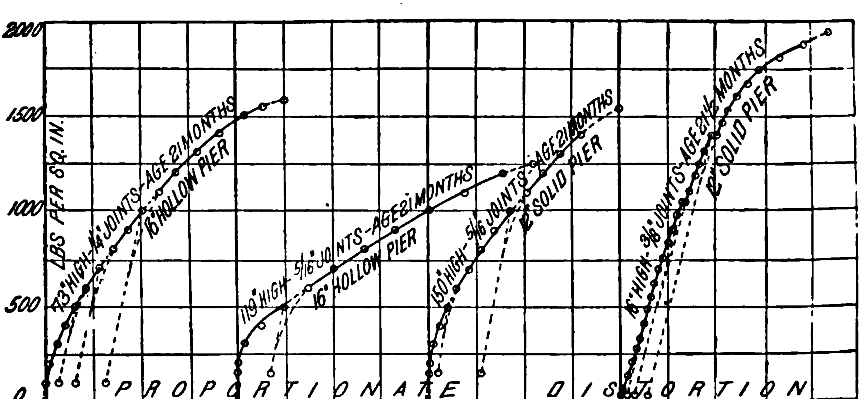

FIG. 596.—Strength of Common-brick Piers with Rosendale-cement-mortar Joints, 1 C. : 2 S. (*Wat. Ars. Rep.* 1883.)

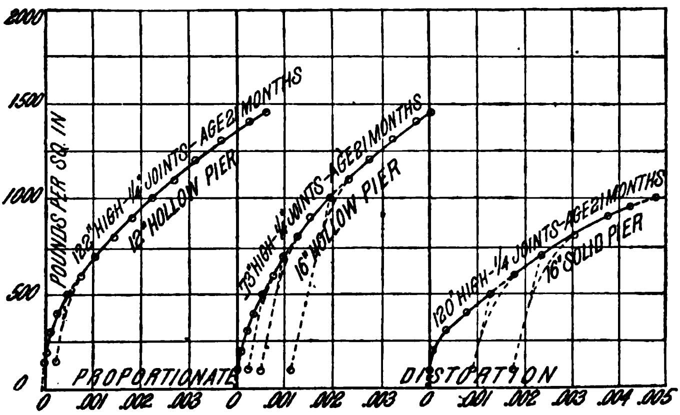

FIG. 597.—Strength of Common-brick Piers with Rosendale-cement-mortar Joints, 1 C. : 2 S. (*Wat. Ars. Rep.* 1883.)

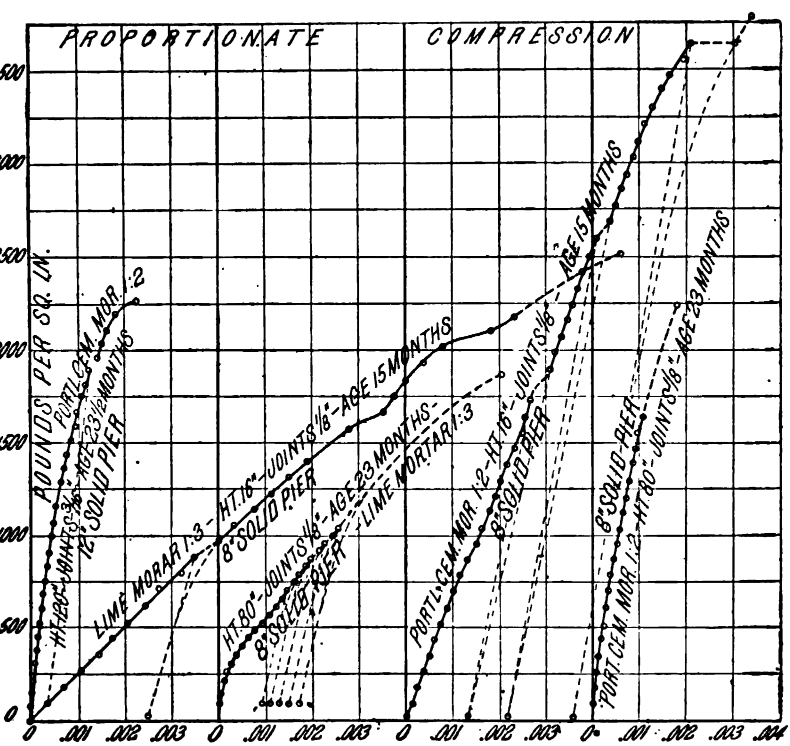

FIG. 598.—Strength of Face-brick Piers with Portland-cement- and Lime-mortar Joints. (*Wat. Ars. Rep.* 1883.)

The ultimate strength is raised to 1650 lbs. with the Rosendale, and to 1450 lbs. per square inch when using the Portland cement, while the modulus of elasticity is also greatly increased, especially under the higher loads.

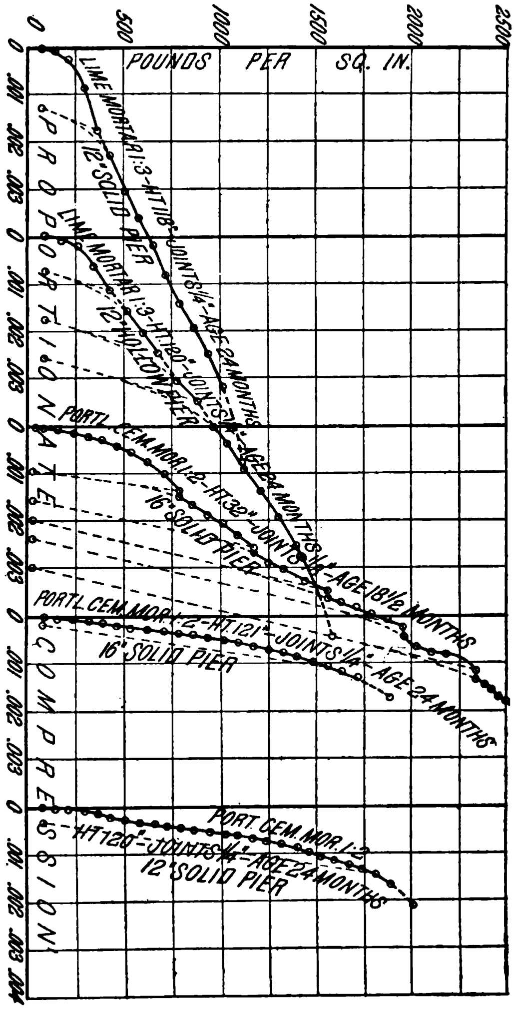

Fig. 599.—Strength of Common-brick Piers with Various Mortars. (Wat. Ars. Rep. 1883.)

The effect of height, as related to breadth, on the crushing strength of brick is shown in Fig. 591, where results are given for bricks crushed flatwise in columns of one, two, three, four, and five bricks high, with plaster-of-paris beds. This curve is quite similar to that in Fig. 17, p. 32.

The remaining diagrams, given in Figs. 592 to 600, showing the strength

of brick columns are thought to be self-explanatory and therefore need no further comment here. The original publications, furthermore, are generally accessible in this country.

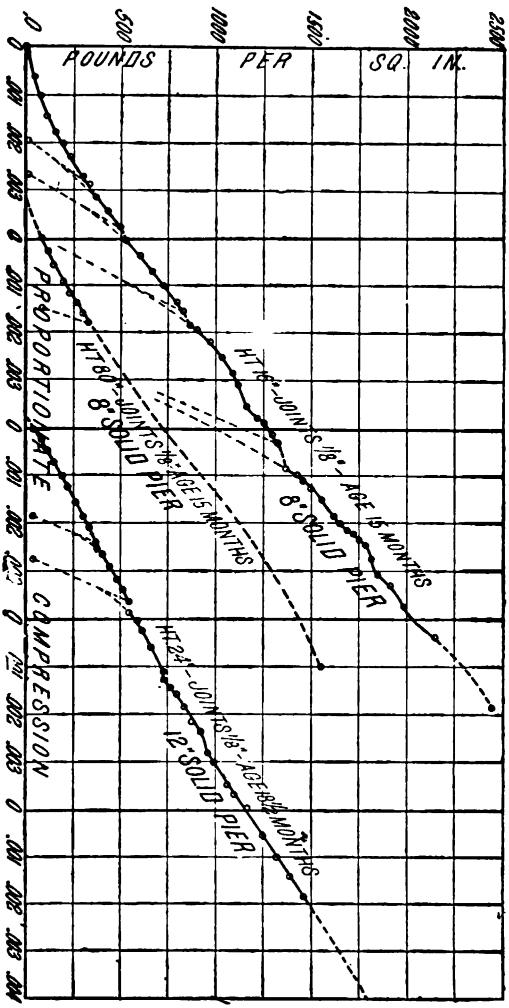

FIG. 600.—Strength of Brick Piers with Lime-mortar Joints. (*Wat. Ars. Rep.* 1883.)

Table XLIV contains a record of tests on some of the best building brick, as made by the hydraulic dry-press method, the tests in crushing having been made upon the bricks flatwise. This gives results about 25 per cent greater than if the tests were made on cubical forms, and 40 per cent greater than if the brick had been crushed edgewise, as was the case with the paving-brick tests on which are recorded in Table XLV.

TABLE XLIV.—COMPARISON OF TRANSVERSE AND CRUSHING TESTS OF BRICK.

(From Watertown Arsenal Tests for 1894.)

Transverse Test No.	Kind of Brick.	Modulus of Rupture in Transverse Tests, Pounds per Square Inch. $f=\frac{3}{2}\frac{Wl}{bh^2}$.	Mean Crushing Strength Flatwise, Pounds per Square Inch.	Ratio: $\frac{\text{Crushing}}{\text{Transverse}}$.
212	From Hydraulic Pressed Brick Co., St. Louis	754	10,350	13.8
213		833	17,698	21.2
214		829	8,495	10.2
215		868	10,483	12.1
216		604	5,573	9.2
223	From Hydraulic Pressed Brick Co., Chicago	455	5,596	12.3
224		808	8,271	10.6
226	From Hydraulic Pressed Brick Co., Omaha	1,244	13,506	10.9
225	From Northern Hydraulic Pressed Brick Co., Minneapolis	455	6,583	14.4
206	From Eastern Hydraulic Pressed Brick Co., Philadelphia	936	12,823	13.7
207		1,232	13,052	10.6
208		1,066	15,633	14.6
209		756	12,196	16.1
210		1,038	12,445	11.9
211		974	12,866	13.2
217	rom Philadelphia and Boston Face-brick Co., Boston	785	8,848	11.3
218		1,043	11,867	11.4
219		741	7,778	10.5
220		568	3,093	5.4
221		858	8,217	9.6
222		358	2,686	7.5

438. Results of Tests of Paving-brick.—Table XLV contains the results of tests made by the author on paving-brick in accordance with the methods he has established and described in Chap. XXII. The brick intended for the cross-breaking and for the crushing tests are ground first on one flat side to obtain a true plane of reference, and then on the opposite edges to true parallel planes. This grinding is done for him at a regular stone (marble) works, and there seems to be little difficulty in obtaining satisfactory results. The knife-edge bearings used in the cross-breaking tests are somewhat rounded, but are not cushioned.

The ends of brick which have been broken across are then used for the crushing test. The crushing force is applied edgewise, using ordinary tarboard as a cushioning material. Plain steel surfaces are better if they are perfectly true. One of the bearings should be adjustable or have a ball-and-socket support. The specimen must also be placed exactly in the axis of the

TABLE XLV.—TESTS OF PAVING-BRICK MADE BY THE AUTHOR.

(Private Records.)

Mark.	Number of Tests Averaged.	Modulus of Rupture in Cross-breaking Edgewise in Pounds per Square Inch. $f=\frac{3wl}{2bh^2}$.	Crushing Strength Edgewise in Pounds per Square Inch.	Impact Test. (See Art. 337, p. 457.) Loss of Weight of Brick, Per Cent.	Loss of Weight of Granite Blocks, Per Cent.	Ratio: $\frac{\text{Loss of brick}}{\text{Loss of granite}}$	Absorption Test. Percentage of Water by Weight.
A	12	1,369	4,885	9.84	2.34	4.2	5.32
B	5	1,495	4,974	13.82	2.78	4.97	4.1
C	6	2,808	9,890	12.15	2,5	4.85	0.64
D	3	3,032	16,140	16.42	3.6	4.56	1.36
E	4	2,620	12,330	13.98	3.6	3.88	3.40
F	3	2,734	15,155	14.34	3.6	3.98	1.12
G	3	2,835	12,040	26.54	3.6	7.37	0.87
H	3	2,835	17,500	19.17	3.6	5.32	0.64
I	3	2,825	17,480	11.04	3.6	3.07	1.12
J	3	2,100	18,150	18.38	3.6	5.11	0.83
K	5	2,570	20,420	26.67	3.38	7.89	0.55
L	5	2,152	15,580	13.0	3.2	4.06	2.20
M	18	2,401	13,866	15.7	1.8	8.7	0.79
N	6	2,635	15,360	16 8	4.1	4.0	2.79
O	6	2,208	13,300	15.3	5.0	3.1	2.92
P	5	2,674	16,830	8.65	2.45	3.55	1.29
Q	6	2,320	14,420	14.53	3.41	4.26	0.67
R	4	3,110	20,802	19.5	2.5	7.8	0.5
S	4	1,780	11,037	13.48	3.03	4.45	6.61
T	5	2,930	13,260	11.37	3.89	2.92	3.08
U	3	2,570	10,400	15.1	2.21	6.83	2.2
V	5	2,640	7,830	34.1	2.21	15.4	1.1

machine. Failure should come suddenly with a loud report, with little or no previous spalling of the specimen.

The impact tests were made in a tumbler, or rattler, made up by lining an oil-barrel with steel strips and mounting it on trunnions. Standard Missouri granite blocks, rectangular in form, and weighing about the same as a paving-brick (6 lbs.) are obtained in quantities, specially prepared for these tests. There were always five of these, freshly cut (not sawed), put in the rattler along with the brick, and the rattler run at 30 to 40 revolutions per minute for 15 to 30 minutes. The loss in weight of the brick is then found as compared with the loss in weight of the granite blocks. It has been customary to add from 5 to 10 cast-iron bricks, having rounded edges, specially cast for the purpose, these also weighing 6 pounds each. Evidently such a test is in no sense an abrasion test, but strictly a test for shock resistance, or for resistance to impact.

The absorption test has been made by drying 24 hours on top of boilers and then soaking 24 hours. While these intervals are not long enough to give absolute results, they serve very well for commercial purposes. The bricks which have been through the rattler are used for this test, as their glazed surfaces are then largely removed.

TABLE XLVI.—TESTS OF BUILDING-BRICK AT THE WATERTOWN ARSENAL.

(*Rep.* 1894.)

Description of Brick.	Compression Test.		Cross-breaking Strength in Pounds per Square Inch.	Shearing Strength in Pounds per Square Inch.	Percentage of Absorption	
	Direction of Loading.	Crushing Strength in Pounds per Square Inch.	$f = \frac{3}{2} \cdot \frac{Wl}{bh^2}$		By Weight.	By Volume.
HYDRAULIC PRESS BRICK CO.—ST. LOUIS, MO.						
Medium red	Flatwise	5,266	....	....	18.0	31.5
Dark red	"	10,284	754			
Paving stock	"	17,558	....	....	10.1	20.0
Paving stock	Edgewise	5,992	....	....	10.1	20.0
No. 6 stock, dark red	Flatwise	10,643	....	1011		
No. 10 stock, dark red	"	17,023	833			
No. 500 stock, buff, speckled	"	8,815	829			
No. 503 stock, light chocolate	"	8,620				
No. 504 stock, light chocolate with dark speckles	"	11,432	868			
No. 509 stock, dark buff with darker speckles	"	8,907				
No. 510 stock, buff with dark speckles	"	6,388	604	....	8.0	16.1
No. 511 stock, light buff	"	8,144	....	642	9.6	19.0
Brown	"	8,861	....	1047	15.4	28.1
CHICAGO HYDRAULIC PRESS BRICK CO.—CHICAGO, ILL.						
Brown	Flatwise	3,779	308	....	14.6	27.7
Red	"	5,589	455	784	14.8	27.9
Red	Edgewise	5,192				
OMAHA HYDRAULIC PRESS BRICK CO.—OMAHA, NEB.						
Shade No. 5	Flatwise	13,511				
Shade No. 7	"	12,907	....	.....	11.4	22.2
Shade No. 6	"	13,506	1244			
NORTHERN HYDRAULIC PRESS BRICK CO.—MINNEAPOLIS, MINN.						
Dark red	Flatwise	7,509	....	....	14.8	27.4
Dark red	"	6,814	455	714		
BROOKE TERRA-COTTA CO.—LAZEARVILLE, W. VA.						
No. 4, dark buff	Flatwise	20,616	....	....	7.6	14.6
No. 5, medium dark buff	"	10,950	....	....	9.1	17.6
No. 10, light buff	"	18,574	....	....	6.0	12.6
FINDLAY HYDRAULIC PRESS BRICK CO.—FINDLAY, OHIO.						
No. 12, dark red	Flatwise	9,686				
No. 18, dark red	"	12,372				
No. 14, dark red	"	11,201				

TESTS OF BUILDING-BRICK AT THE WATERTOWN ARSENAL—*continued.*

Description of Brick.	Compression Test.		Cross-breaking Strength in Pounds per Square Inch.	Shearing Strength in Pounds per Square Inch.	Percentage of Absorption	
	Direction of Loading.	Crushing Strength in Pounds per Square Inch.	$f = \frac{3}{2} \cdot \frac{Wl}{bh^2}$		By Weight.	By Volume.
EASTERN HYDRAULIC PRESS BRICK CO.—PHILADELPHIA, PA.						
Shade 200, light buff color.....	Flatwise	15,285	936	....	6.9	14.5
Shade 210, slightly darker than shade 200 ..	"	13,292	1232	1767		
Shade 210..................	Edgewise	9,819				
Shade 220, buff..............	Flatwise	15,374	1066	1097	5.5	11.6
Shade 300, buff, darker........	"	12,671	756	....	7.9	16.1
Shade 300..................	Edgewise	9,278				
Shade 390, gray..............	Flatwise	13,059	1038	988	7.1	14.5
Shade 410, light chocolate......	"	15,081				
Shade 410..................	Edgewise	9,945	....	....	5.5	11.5
Shade 400..................	Flatwise	12,866	974			
PHILADELPHIA AND BOSTON FACE-BRICK CO. — BOSTON, MASS.						
Salmon color................	Flatwise	3,896	....	....	19.0	32.6
Light red..................	"	8,487	785	....	11.0	21.6
Light red..................	Edgewise	5,877				
Dark red..................	Flatwise	10,942	1043	....	10.0	20.1
Chocolate-brown.............	"	7,774	741			
Chocolate-brown.............	Edgewise	17,023				
Cream color................	Flatwise	3,161	568	....	18.1	31.0
Buff......................	"	8,946	858			
Buff......................	Edgewise	4,756				
Gray......................	Flatwise	3,070	358	536	15.2	27.1

In Table XLVI are given the more significant results of a very careful series of tests on building brick, these being a part of an elaborate series of tests on building materials begun in 1894, and still in progress. These brick are supposed to represent the better grades of building brick on the market in different parts of this country. The compression tests were made by bedding the pressed surfaces in plaster of Paris. A great difference will be observed between the crushing strength flatwise and edgewise. The strength of these brick, when tested singly, should be compared with the strength of brick piers, with various mortars, as shown in Figs. 590 to 600.

CHAPTER XXXII.

EXPERIMENTAL VALUES OF THE STRENGTH OF TIMBER.

439. The Mechanical Tests of the U. S. Timber Investigations.—Although timber is the oldest and still the most universally used of all structural material, no rational determination of the laws controlling the strength of timber has been attempted until within a few years. Bauschinger made a few experiments in 1882, and pointed the way to a thorough study of timber which since 1890 has been conducted by the U. S. Government.* These investigations are still incomplete, but they already furnish a vast amount of valuable information, a part of which has been published in bulletins and circulars issued by the Forestry Division (Dr. B. E. Fernow, Chief) of the U. S. Agricultural Department from time to time. The following direct quotations in this chapter are taken from Forestry Circular No. 15, 1897:

"The superiority of the data obtained in these investigations lies in (1) the correct identification of the material, it being collected by a competent botanist in the woods; (2) selection of representative trees with record of age, development, place, and soil where grown, etc.; (3) determination of moisture conditions, specific gravity, and record of position in the tree of the test-pieces; (4) large number of trees and of test-pieces from each tree (see Table XLVII); (5) employment of large- and small-sized test material from the same trees; (6) uniformity of method for an unusually large number of tests.

"The entire work of the mechanical test series carried on through nearly six years intermittently, as funds were available, comprises so far 32 species with 300 test trees, furnishing over 6000 test-sticks and about 40,000 tests in all.†

"In addition to the material for mechanical tests, about 20,000 pieces of material for physical examination from 780 trees (including the 300 trees used in mechanical tests) have been collected to determine structure, character of growth, specific gravity of green and dry wood, shrinkage, moisture conditions, and other properties and behavior.

* For a short general description of these investigations see Art. 340, p. 462.

† These tests have all been conducted under the direction of the author in his laboratory at St. Louis, Mo.

TABLE XLVII.—AN ACCOUNT OF THE MATERIAL OPERATED UPON (1891–1896) IN THE U. S. TIMBER INVESTIGATIONS.

(From *U. S. Forestry Circular*, No. 15.)

Number.	Name of Species.	Number of Trees.	Number of Tests.	Average Specific Gravity of Dry Wood.†	Localities, and Number of Trees from Each.
1	Long-leaf pine (*Pinus palustris*)	* 68	6478	.61	Alabama—coast plain (22)*, uplands (6), hill district (6): Georgia—undulating uplands (6): South Carolina — coast plain (7); Mississippi—low coast plain (2); Louisiana—low coast plain, gravelly soil (7), sandy loam (6); Texas—low-coast plain (6)
2	Cuban pine (*Pinus heterophylla*)	12	2118	.63	Alabama—coast plain (6); Georgia—uplands (1); South Carolina—coast (5)
3	Short-leaf pine (*Pinus echinata*)	22	1831	.51	Alabama — uplands (4); Missouri — low hilly uplands (6); Arkansas—low hilly uplands (6); Texas—uplands (6)
4	Loblolly-pine (*Pinus tæda*)	32	3335	.53	Alabama—mountainous plateau (8), low coast plain (6); Arkansas—level flood plain (5); Georgia—level coast plain (6); South Carolina—low coast plain (7)
5	White pine (*Pinus strobus*)	17	540	.38	Wisconsin—clay uplands (5), sandy soils (4), sandy loam (5); Michigan—level drift-lands (3)
6	Red pine (Norway pine) (*Pinus resinosa*)	8	412	.50	Wisconsin—drift (5); Michigan—(3)
7	Spruce-pine (*Pinus glabra*)	4	696	.44	Alabama—low coast plain
8	Bald cypress (*Taxodium distichum*)	20	3396	.46	South Carolina—pine-barren (6), river-bottom (4); Louisiana—coast plain, border of lake (4); Mississippi—Yazoo bottom (3), upland (3)
9	White cedar (*Chamæcyparis thyoides*)	4	354	.37	Mississippi—low plain (4)
10	Douglas spruce (Oregon fir) (*Pseudotsuga taxifolia*) (*douglasii*)	....	225	.51	
11	White oak (*Quercus alba*)	12	1009	.80	Alabama—ridges of Tennessee Valley (5); Mississippi—low plain (7)
12	Overcup-oak (*Quercus lyrata*)	10	911	.74	Mississippi — low plain (7); Arkansas — Mississippi bottoms (3)
13	Post-oak (*Quercus minor*)	8	256	.80	Alabama—Tennessee Valley (5); Arkansas —Mississippi bottom (3)
14	Cow-oak (*Quercus michauxii*)	11	935	.74	Alabama—Tennessee Valley (4): Arkansas —Mississippi bottoms (3); Mississippi—low plain (4)

* Sixteen of these were bled trees, to study the effects of boxing.

† The specific gravity here presented is, for all but 8 of the conifers, that of the test-pieces only, and is not an average for the material on the whole.

AN ACCOUNT OF THE U. S. TIMBER INVESTIGATIONS—*continued.*

Number.	Name of Species.	Number of Trees.	Number of Tests.	Average Specific Gravity of Dry Wood.*	Localities, and Number of Trees from Each.
15	Red oak (*Quercus rubra*)	5	299	.78	Alabama—Tennessee Valley (5)
16	Texan oak (Southern red oak) (*Quercus texana*)	5	479	.73	Arkansas—Mississippi bottom (2); Mississippi—low plain (3)
17	Yellow oak (black) (*Quercus velutina*)	5	222	.72	Alabama—Tennessee Valley (5)
18	Water-oak (*aquatica*) (*Quercus nigra*)	4	132	.73	Mississippi—low plain (4)
19	Willow-oak (*Quercus phellos*)	12	649	.72	Alabama—Tennessee Valley (5); Arkansas—Mississippi bottom (3); Mississippi—low plain (4)
20	Spanish oak (*Quercus digitata*)	11	1035	.78	Alabama—Tennessee Valley (5); Arkansas—Mississippi bottom (3); Mississippi—low plain (3)
21	Shagbark (shellbark) hickory (*Hicoria ovata*)	6	794	.81	Mississippi—alluvial plain (3), limestone (3)
22	Mockernut hickory (white) (*Hicoria alba*)	4	300	.85	Mississippi—low plain
23	Water-hickory (*Hicoria aquatica*)	2	197	.73	Mississippi—low plain
24	Bitternut hickory (*Hicoria minima*)	4	100	.77	Mississippi—low plain
25	Nutmeg-hickory (*Hicoria myristicæformis*)	3	294	,78	Mississippi—low plain
26	Pecan (*Hicoria pecan*)	2	172	.78	Mississippi—low plain
27	Pignut hickory (*Hicoria glabra*)	3	84	.89	Mississippi—low plain
28	White elm (*Ulmus americana*)	2	91	.54	Mississippi bottom
29	Cedar elm (*Ulmus crassifolia*)	3	201	.74	Arkansas bottom
30	White ash (*Fraxinus americana*)	3	476	.62	Mississippi bottom
31	Green ash (*viridis*) (*Fraxinus lanceolata*)	1	45	.62	Mississippi bottom
32	Sweet-gum (*Liquidambar styraciflua*)	7	508	.59	Arkansas—bottom (3); Mississippi—low plain (4)

* The specific gravity here presented is, for all but 8 of the conifers, that of the test-pieces only, and is not an average for the material on the whole.

440. The Investigations still in Progress.—"As will be observed, some species, notably the Southern pines, have been more fully investigated, and the results on these (which have been published in more detail in Circular No. 12) may be taken as authoritative. With those species of which only a

small number of trees have been tested, this can be claimed only within limits and in proportion to the number of tests.

"The great variation in strength which is noticeable in timber of the same species makes it necessary to accept with caution the result of a limited number of tests as representing the average for the species, for it may have happened that only superior or only inferior material has come to test. Hence we would not be entitled to conclude that pignut hickory is 14 per cent stronger than shellbark, as it would appear in the tables, for the 30 test-sticks of the former may easily have been superior material. Only a detail examination of the test-pieces or a fuller series of tests would enlighten us as to the comparative value of the results.

"The following data, therefore, are not to be considered as in any sense final values for the species except where the number of trees and tests is very large. The variation in strength, as will be seen from the tables, in wood of the virgin forest is in some species so great that by proper inspection and selection values differing by 25 to 50 per cent may be obtained from different parts of the same tree, and values differing 100 to 200 per cent within the same species. These differences have all their definite recognizable causes, to find and formulate which is the final aim of these investigations.

"The tests are intentionally not made on selected material (except to discard absolutely defective pieces), but on material as it comes from the trees, so as to arrive at an average statement for the species, when a sufficient number of trees has been tested. How urgent is the need for data of inspection as above indicated will appear from the wide range of results recorded.

"To enable any engineer to use the data here given with due caution and judgment, not only the ranges of values and the average of all values obtained, but also the proportion of tests which came near the average values has been stated, as well as the average result of the highest and lowest values of 10 per cent of the tests. With this information and a statement of the actual number of tests involved, the comparative merit of the stated values can be judged. With a large number of tests, to be sure, it is more likely that an average value of the species has been found. The actual test results have been rounded off to even hundreds in the tables."

441. The Moisture Factor in timber was described in Articles 192 and 202. It has been determined by cutting a thin disc from across the entire section of the test-stick near the place of failure and finding its weight as first taken and after drying at 220° F. In the following tables all values have been reduced to a standard moisture of 12 per cent,* which may be regarded as that of dry timber out of doors. *With all the species tested the strength at 12 per cent moisture is some 75 per cent stronger than the same*

* The author is responsible for the reduction of the results on the Southern pines from 15 per cent to 12 per cent moisture to bring them into harmony with the other test results. The former was used as the standard of reference at first, but it was afterwards decided this was too high for well-seasoned timber. (See Forestry Circular No. 12 for the strength-moisture curves for the Southern pines.)

sticks are either green or when wet through after seasoning. In fact, it has been shown that water reabsorbed after drying (which is the same as seasoning) has the same weakening effect as the original sap. This is manifest from Fig. 601, which contains the results of a series of tests on

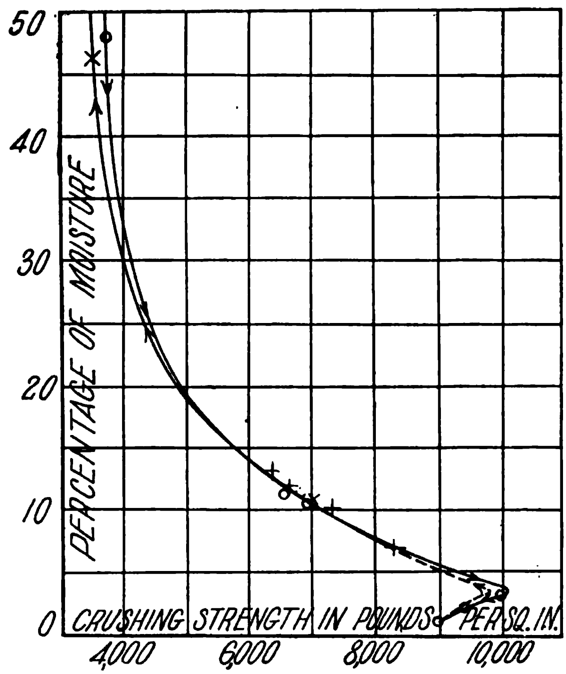

FIG. 601.—Showing Variation of Strength of Short-leaf Pine. Sap-wood, with Varying Percentages of Moisture both for Drying and for Reabsorbing Conditions. Tests by the author. Drying conditions marked by a 0 : reabsorbing conditions marked by a ×.

identical material tested at various moisture conditions in drying out to a nearly zero moisture, and again at similar moisture conditions when moisture was uniformly reabsorbed.

It is the absence of any determination of the moisture condition of the test material that vitiates practically all tests of the strength of timber except such as have been made by Bauschinger, Tetmajer, and those here under consideration. Since large timbers require many years to season, or dry, in the open air, while small test sticks dry out very quickly, it is certain that the difference in the moisture conditions will fully explain the marked differences which have been observed in the strength of identical material in different sizes. It is to be hoped that in future all tests of the strength of timber will be so made as to fully reveal this condition as a definite percentage of moisture across the section near the region of failure.

In all the tests made by the author, practically identical strength moduli

were obtained on large and small sizes of the same material when they were all reduced to the same moisture condition and were equally free from defects. Hence tests on small sizes (3 to 4 inches on a side) will give reliable factors to use in actual practice.

As shown in Fig. 602, the increase of strength with diminishing moisture

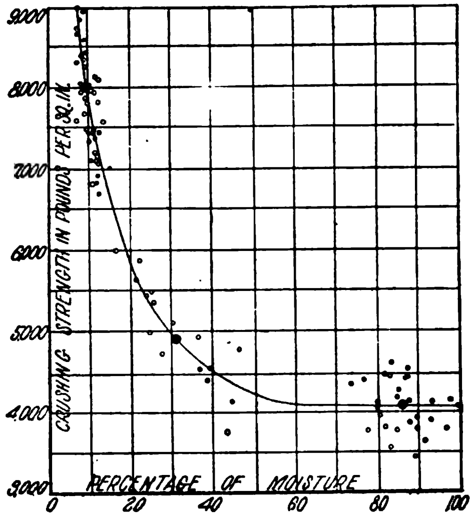

FIG. 602.—Typical Relation between Strength and Moisture of Timber. This diagram shows the relation of crushing strength parallel to the grain to the percentage of moisture for one species of oak. (From the Author's *U. S. Timber-Test Records*.)

does not become apparent until the moisture percentage becomes less than about 40 per cent (and if it were quite evenly distributed it would be at about 33 per cent). For a greater percentage of moisture than this the water fills not only the cell-walls (see Art. 194), but also the cell-cavities or lumina. Since the weakening effect comes only from the wetting of the walls themselves, it follows that after they are fully saturated any excess of water which occupies the cell lumina would be inoperative. No increase of strength is noticeable, therefore, until the cell-walls themselves (the woody fibre) begin to dry out, when the increase of strength becomes very rapid. If this drying action could occur uniformly across the entire cross-section of the specimen, the locus of the strength-moisture relation would be practically two straight lines, one quite straight and parallel to the moisture axis, and the second somewhat convex to the strength axis, and intersecting the former locus at about 33 per cent moisture. The locus becomes a continuous curve when the outer parts dry very much more rapidly than the inner parts, as is the case, of necessity, in all processes of drying. To avoid this mixed condition it is necessary to make the tests on material absolutely green * or uniformly

* The "green" sticks of the U. S. tests were placed, after sawing, in a "wet room," where the air was kept at the point of saturation, or as nearly so as possible.

dry (but still containing from 10 to 15 per cent of atmospheric moisture). Timber is never uniformly half dry.

TABLE XLVIII.—STRENGTH OF AMERICAN TIMBER. CONDENSED RESULTS OF THE U. S. TIMBER TESTS. ALL VALUES REDUCED TO A STANDARD MOISTURE OF 12 PER CENT OF THE DRY WEIGHT.

(Compiled from the tables in *U. S. Forestry Circular*, No. 15.)

Species.	Number of Trees Tested.	Number of Sticks Averaged.	Specific Gravity.	Weight per Cubic Foot.	Cross-bending Tests. Apparent Elastic Limit.	Cross-bending Tests. Ultimate Strength.	Cross-bending Tests. Modulus of Elasticity.	Crushing Endwise.	Crushing across Grain.	Shearing along the Grain.
					lb. sq. in.	lb. sq. in.	lb. sq. in.	lb. sq. in.	lb. sq. in.	lb. sq. in.
1. Long-leaf pine......	68	1230	.61	38	10,000	12,600	2,070,000	8.000	1.260	835
2. Cuban "	16	410	63	39	11,100	13,600	2,370,000	8,700	1,200	770
3. Short-leaf "	22	330	.51	32	7,800	10,100	1,680,000	6,500	1,050	770
4. Loblolly- "	32	660	.53	33	9,200	11,300	2,050,000	7,400	1,150	800
5. White "	6	130	.38	24	6,400	7,900	1,390,000	5,400	700	400
6. Red "	8	100	.50	31	7,700	9,100	1,620,000	6,700	1,000	500
7. Spruce- "	3	170	.62	39	8,400	10,000	1,640,000	7,300	1,200	800
8 Bald cypress.......		655	.46	29	6,600	7,900	1,290,000	6,000	800	500
9. White cedar.......		87	.37	23	5,800	6,300	910,000	5,200	700	400
10. Douglas spruce....		41	.51	32	6,400	7,900	1,680,000	5,700	800	500
11. White oak.........	14	218	.80	50	9,600	13,100	2,090,000	8,500	2,200	1,000
12. Overcup-oak.......	10	216	.74	46	7,500	11,300	1,620,000	7,300	1,900	1,000
13. Post- "	5	49	.80	50	8,400	12,300	2,030,000	7,100	3,000	1,100
14. Cow- "	11	256	.74	46	7,600	11,500	1,610,000	7,400	1,900	900
15. Red "	6	57	.72	45	9,200	11,400	1,970,000	7,200	2,300	1,100
16. Texan "	3	117	.73	46	9,400	13,100	1,860,000	8,100	2,000	900
17. Yellow "	5	40	.72	45	8,100	10,800	1,740,000	7,300	1,800	1,100
18. Water- "	8	81	.73	46	8,800	12,400	2,000,000	7,800	2,000	1,100
19. Willow- "	12	153	.72	45	7,400	10,400	1,750,000	7,200	1,600	900
20. Spanish "	5	251	.73	46	8,600	12,000	1,930,000	7,700	1,800	900
21. Shagbark hickory..	6	137	.81	51	11,200	16,000	2,390,000	9,500	2,700	1,100
22. Mockernut "	4	75	.85	53	11,700	15,200	2,320,000	10,100	3,100	1,100
23. Water- "	2	14	.73	46	9,800	12,500	2,080,000	8,400	2,400	1.000
24. Bitternut "	4	25	.77	48	11,100	15,000	2,280,000	9.600	2,200	1.000
25. Nutmeg- "	3	72	.78	49	9,300	12,500	1,940,000	8,800	2,700	1,100
26. Pecan "	2	37	.78	49	11,600	15,300	2,530,000	9.100	2,800	1,200
27. Pignut "	3	30	.89	56	12,600	18,700	2,730,000	10,900	3,200	1,200
28. White elm.........	2	18	.54	34	7,300	10,300	1,540,000	6,500	1,200	800
29. Cedar- "	3	44	.74	46	8,000	13,500	1,700,000	8,000	2,100	1,300
30. White ash.........	3	87	.62	39	7,900	10,800	1,640,000	7,200	1,900	1,100
31. Green "	1	10	.62	39	8,900	11,600	2,050,000	8,000	1,700	1,000
32. Sweet-gum........	7	118	.59	37	7,800	9,500	1,700,000	7,100	1,400	800

TABLE XLIX.—CRUSHING STRENGTH OF TIMBER, ENDWISE, IN POUNDS PER SQUARE INCH REDUCED TO THE STANDARD PERCENTAGE OF MOISTURE.

(From *U. S. Forestry Circular*, No. 15.)

Species.	Per cent of Moisture to which Results are Reduced.	Number of Tests Averaged.	Average of all Tests.	Average of Highest Ten Per cent.	Average of Lowest Ten Per cent.	Highest Single Result.	Lowest Single Result.	Proportion of all Tests within 10% of Average.	Proportion of all Tests within 25% of Average.
1. Long-leaf pine.......	15	1,230	6,900	8,600	5,700	11,900	3,400	0.53	0.90
2. Cuban "	15	410	* 7,900	9,500	6,500	10,600	2,800	.61	.93
3. Short-leaf "	15	330	5,900	7,600	4,800	8,500	4,500	.47	.90
4. Loblolly- "	15	660	6,500	8,700	5,400	11,200	3,900	.49	.84
5. White "	12	130	5,400	6,800	4,000	8,500	3,200	.49	.93
6. Red "	12	100	6,700	8,100	4,900	8,200	4,300	.54	.96
7. Spruce- "	12	170	7,300	8,800	5,600	10,000	4,400	.66	.95
8. Bald cypress.........	12	655	6,000	8,500	4,200	9,900	2,900	.31	.74
9. White cedar.........	12	87	5,200	6,000	4,400	6,200	3,200	.79	.99
10. Douglas spruce......	12	41	5,700	8,100	4,200	8,900	4,100	.28	.65
11. White oak..........	12	218	8,500	11,300	6,300	12,500	5,100	.40	.81
12. Overcup oak.........	12	216	7,300	8,600	6,000	9,100	3,700	.70	.95
13. Post-oak.............	12	49	7,100	8,100	6,000	8,200	5,900	.58	1.00
14. Cow-oak............	12	256	7,400	9,800	5,600	11,500	4,600	.51	.89
15. Red oak.............	12	57	7,200	9,200	5,500	9,700	5,400	.36	.94
16 Texan oak..........	12	117	8,100	9,800	6,900	11,300	5,800	.62	.98
17. Yellow "	12	40	7,300	8,300	5,800	8,600	5,500	.58	1.00
18. Water- "	12	31	7,800	9,000	6,300	9,200	6,200	.75	1.00
19. Willow-"	12	153	7,200	8,700	5,500	11,000	4,200	.51	.88
20. Spanish "	12	251	7,700	9,500	5,100	10,600	3,700	.61	.94
21. Shagbark hickory...	12	137	6,500	10,900	7,500	13,700	5,800	.79	.97
22. Mockernut " ...	12	75	10,100	11,600	8,000	12,200	6,200	.65	.99
23. Water- " ...	12	14	8,400	9,600	7,000	10,000	6,700	.71	1.00
24. Bitternut " ...	12	25	9,600	11,200	7,800	11,500	7,300	.60	1.00
25. Nutmeg- " ...	12	72	8,800	11,000	7,100	12,300	6,400	.79	.97
26. Pecan ...	12	37	9,100	10,400	7,300	10,000	5,800	.51	.95
27. Pignut " ...	12	80	10,900	12,700	8,900	13,000	8,700	.72	1.00
28. White elm...........	12	18	6,500	8,800	5,000	8,800	4,900	.28	.88
29. Cedar "	12	44	8,000	10,100	6,500	10,600	6,200	.66	.95
30. White ash	12	87	7,200	8,700	5,700	9,600	5,000	.48	.96
31. Green "	12	10	8,000	9,800	6,600	9,800	6,600	.29	1.00
32. Sweet-gum..........	12	118	7,100	8,500	5,600	8,900	4,600	.60	.97

* These results should be increased from 12 to 15 per cent to reduce them to a tandard moisture of 12 per cent. See table on p. 670 for results corrected to 12 per cent moisture.

TABLE L.—STRENGTH OF GREEN TIMBER IN COMPRESSION ENDWISE.

This timber contained more than forty per cent of moisture.

(From *U. S. Forestry Circular*, No. 15.)

Species.	Number of Sticks Tested.	Average Compressive Strength in Pounds per Square Inch.	Highest Single Result, Pounds per Square Inch.	Lowest Single Result, Pounds per Square Inch.
1. Long-leaf pine	86	4,300	7,300	2,800
2. Cuban "	38	4,800	6,100	3,500
3. Short-leaf "	8	3,300	4,000	3,000
4. Loblolly- "	69	4,100	5,500	2,600
7. Spruce- "	71	3,900	4,700	2,800
8. Bald cypress	280	4,200	8,200	1,800
9. White cedar	34	2,900	3,400	2,300
11. White oak	25	5,300	7,000	3,200
12. Overcup-oak	45	3,800	4,900	2,800
14. Cow-oak	58	3,800	4,900	2,300
16. Texan oak	39	5,200	6,000	3,100
19. Willow-oak	49	3,800	5,500	2,300
20. Spanish oak	52	3,900	5,100	2,500
21. Shagbark hickory	22	5,700	6,900	3,500
22. Mockernut "	18	6,100	7,200	4,500
23. Water- "	4	5,200	5,600	4,700
25. Nutmeg- "	26	4,500	5,500	3,700
26. Pecan "	4	3,600	3,800	3,300
27. Pignut "	5	5,400	6,200	4,700
32. Sweet-gum	6	3,300	3,600	3,000

442. Other Special Investigations.—In addition to regular tests the results of which are summarized in Tables XLVIII to LIII, the following special investigations have been in progress, the mechanical tests connected therewith being under the author's supervision. Some of the conclusions stated below must be accepted as provisional, pending further experiments along these lines.

1. *The Effect of "Bleeding"* (boxing, or tapping, for turpentine) long-leaf pine-trees on the qualities of the lumber subsequently cut from the same. This investigation included 1300 mechanical tests on bled timber taken from two sites, one where the trees had been bled and abandoned for five years, and the other freshly bled and abandoned. These results were compared with the regular tests on unbled timber. In addition 300 chemical analyses were made on bled and unbled timber. These investigations proved beyond a doubt that *the "bleeding" of long-leaf pine timber has absolutely no effect on its strength, and probably none on its value or life when exposed to the weather.* See Forestry Bulletin No. 8. (This conclusion is final.)

2. *Influence of Size on the Strength of Beams.*—This investigation included 433 tests in all. Large beams were first tested to rupture, and then small

TABLE LI.—STRENGTH OF TIMBER IN CROSS-BREAKING. THE MODULUS OF RUPTURE REDUCED TO THE STANDARD PERCENTAGES OF MOISTURE. POUNDS PER SQUARE INCH.

(From *U. S. Forestry Circular*, No. 15.)

Species.	Per Cent of Moisture to which the Results are Reduced.	Number of Tests Averaged.	Average Modulus of Rupture.	Average of Highest Ten per Cent.	Average of Lowest Ten Per Cent.	Highest Single Result.	Lowest Single Result.	Proportion of Results within 10% of Average.	Proportion of Results within 25% of Average.
1. Long-leaf pine........	15	1,160	*10,900	14,200	8,800	17,800	3,300	0.41	0.84
2. Cuban "	15	390	*11,900	14,600	8,800	17,000	2,900	.46	.88
3. Short-leaf "	15	330	*9,200	12,400	7,000	15,300	5,000	.40	.79
4. Loblolly- "	15	650	*10,100	13,100	8,100	14,800	3,900	.44	.84
5. White "	12	120	7,900	10,100	5,000	11,100	4,600	.43	.81
6. Red "	12	95	9,100	12,300	4,900	12,900	3,100	.28	.60
7. Spruce- "	12	170	10,000	13,600	5,800	16,300	3,100	.43	.81
8. Bald cypress.........	12	655	7,900	11,700	5,000	14,800	2,300	.25	.69
9. White cedar..........	12	87	6,300	8,400	4,000	9,100	3,500	.32	.78
10. Douglas spruce......	12	41	7,900	12,000	4,100	13,000	3,800	.22	.58
11. White oak...........	12	218	13,100	18,500	7,600	20,300	5,700	.39	.75
12. Overcup-oak.........	12	216	11,300	14,900	6,300	19,600	4,900	.47	.81
13. Post- "	12	49	12,300	15,300	7,400	16,400	5,100	.47	.92
14. Cow- "	12	256	11,500	12,500	6,500	23,000	3,300	.32	.68
15. Red "	12	57	11,400	15,400	9,100	16,500	5,700	.46	.84
16. Texan "	12	117	13,100	16,900	10,000	19,500	8,200	.64	.86
17. Yellow "	12	40	10,800	14,600	5,700	15,000	5,100	.28	.65
18. Water- "	12	31	12,400	15,700	7,200	16,000	5,800	.40	.76
19. Willow- "	12	153	10,400	13,800	5,400	16,000	3,300	.33	.70
20. Spanish "	12	257	12,000	15,600	6,900	17,300	5,000	.40	.72
21. Shagbark hickory....	12	187	16,000	20,300	9,400	23,300	5,700	.46	.84
22. Mockernut "	12	75	15,200	19,700	7,900	20,700	5,300	.45	.78
23. Water- "	12	14	12,500	17,300	5,400	18,000	5,300	.21	.64
24. Bitternut "	12	25	15,000	19,300	8,700	19,500	7,000	.28	.60
25. Nutmeg- "	12	72	12,500	15,600	8,100	16,600	6,700	.40	.88
26. Pecan "	12	37	15,300	18,100	10,300	18,300	5,600	.38	.95
27. Pignut "	12	30	18,700	24,300	11,500	25,000	11,100	.43	.77
28. White elm...........	12	18	10,300	13,600	7,300	14,000	7,300	.44	.72
29. Cedar- "	12	44	13,500	17,300	8,500	19,200	6,600	.50	.86
30. White ash...........	12	87	10,800	14,200	6,300	15,000	5,000	.37	.77
31. Green "	12	10	11,600	16,000	5,100	16,000	5,100	.20	.60
32. Sweet-gum	12	118	9,500	12,700	6,000	14,400	5,100	.39	.79

* These results should be increased from 12 to 15 per cent to reduce them to a standard moisture of 12 per cent. See table on p. 670 for results corrected to 12 per cent moisture.

TABLE LII.—ELASTIC LIMIT STRENGTH OF TIMBER IN CROSS-BENDING AND THE MODULUS OF ELASTICITY IN POUNDS PER SQUARE INCH, BOTH REDUCED TO STANDARD PERCENTAGES OF MOISTURE.

(From *U. S. Forestry Circular*, No. 15.)

Species.	Per Cent of Moisture to which Results are Reduced.	Number of Tests Averaged.	Average Modulus of Elasticity.	Average Modulus of Elastic Strength.†	Average of Highest Ten Per Cent.	Average of Lowest Ten Per Cent.	Highest Single Modulus of Rupture.	Lowest Single Modulus of Rupture.	Proportion of Results within 10% of Average.	Proportion of Results within 25% of Average.
1. Long-leaf pine...	15	1,160	1,890,000	8,500	11,100	5,400	13,500	2,400	0.43	0.81
2. Cuban "	15	390	2,300,000	9,500	11,500	5,600	12,900	2,200	.42	.83
3. Short-leaf "	15	* 330	1,600,000	7,200	9,700	4,800	11,900	2,900	.48	.81
4. Loblolly- "	15	650	1,950,000	8,200	10,800	5,400	12,700	3,100	.46	.85
5. White "	12	130	1,390,000	6,400	8,200	4,500	10,000	4,100	.58	.85
6. Red "	12	95	1,620,000	7,700	10,300	4,500	11,300	3,100	.38	.73
7. Spruce- "	12	170	1,640,000	8,400	11,700	5,000	13,700	3,000	.51	.82
8. Bald cypress.......	12	655	1,290,000	6,600	9,900	4,200	12,000	2,200	.25	.66
9. White cedar	12	87	910,000	5,800	7,300	4,000	8,200	3,400	.44	.86
10. Douglas spruce....	12	41	1,680,000	6,400	9,600	3,400	13,700	2,800	.32	.56
11. White oak..........	12	218	2,090,000	9,600	14,100	6,100	15,700	4,400	.37	.73
12. Overcup-oak.......	12	216	1,620,000	7,500	9,500	5,400	11,600	4,000	.47	.91
13. Post- "	12	49	2,030,000	8,400	9,600	6,000	10,600	5,100	.34	.76
14. Cow- "	12	256	1,610,000	7,600	11,600	5,000	14,200	3,400	.50	.95
15. Red "	12	57	1,970,000	9,200	13,600	5,600	14,500	5,100	.15	.49
16. Texan "	12	117	1,860,000	9,400	11,400	7,800	12,009	5,950	.62	.94
17. Yellow "	12	40	1,740,000	8,100	11,100	5,100	11,800	4,900	.35	.75
18. Water- "	12	31	2,000,000	8,803	11,400	5,500	11,800	4,500	.40	.84
19. Willow- "	12	153	1,750,000	7,400	10,000	4,300	13,100	2,700	.42	.81
20. Spanish "	12	257	1,930,000	8,600	11,600	6,600	13,500	5,100	.41	.80
21. Shagbark hickory.	12	187	2,390,000	11,200	14,200	7,700	16,100	5,400	.50	.89
22. Mockernut "	12	75	2,320,000	11,700	14,600	7,800	15,400	4,300	.39	.83
23. Water- "	12	14	2,080,000	9,800	11,800	4,800	11,900	4,100	.21	.86
24. Bitternut "	12	25	2,280,000	11,100	14,000	7,600	14,300	7,500	.44	.84
25. Nutmeg- "	12	72	1,940,000	9,300	11,300	6,400	12,200	4,200	.46	.93
26. Pecan "	12	87	2,580,000	11,600	14,400	7,900	15,000	5,800	.65	.89
27. Pignut "	12	30	2,730,000	12,600	16,400	8,300	17,000	7,400	.40	.83
28. White elm..........	12	18	1,540,000	7,300	9,600	5,400	9,700	5,300	.33	.71
29. Cedar- "	12	44	1,700,000	8,000	10,100	5,800	10,700	4,700	.57	.91
30. White ash..........	12	87	1,640,000	7,900	10,400	5,200	11,500	3,600	.43	.83
31. Green "	12	10	2,050,000	8,900	13,200	3,200	13,200	3,200	.40	.70
32. Sweet-gum	12	118	1,700,000	7,800	10,100	5,100	11,000	3,500	.46	.82

* These results should be increased from 12 to 15 per cent to reduce them to a standard moisture of 12 per cent. See results corrected to 12 per cent moisture in table on p. 670.

† This is the Apparent Elastic Limit Strength found as described in Art. 18, p. 18.

(4-in.) sticks were cut from the uninjured ends, from the top side at one end and from the bottom side at the other end. These results prove the truth of the proposition announced in (3) on page 462. (This conclusion is also probably final.)

TABLE LIII.—STRENGTH OF TIMBER IN **CRUSHING ACROSS THE GRAIN** REDUCED TO STANDARD PERCENTAGES OF MOISTURE, IN **SHEARING WITH THE GRAIN**, IN POUNDS PER SQUARE INCH, THE **SPECIFIC GRAVITY**, AND THE WEIGHT PER CUBIC FOOT, THESE NOT BEING REDUCED TO STANDARD MOISTURE.

(From *U. S. Forestry Circular*, No. 15.)

Species.	Number of Tests Averaged.	Crushing-strength Across the Grain.	Shearing-strength with the Grain.	Average Specific Gravity.	Average Weight per Cubic Foot.
1. Long-leaf pine	1,210	*1,000	700	0.61	38
2. Cuban "	400	1,000	700	.63	39
3. Short leaf "	330	900	700	.51	32
4. Loblolly- "	690	1,000	700	.53	33
5. White "	130	700	400	.38	24
6. Red "	100	1,000	500	.50	31
7. Spruce- "	175	1,200	800	.62	39
8. Bald cypress	650	800	500	.46	29
9. White cedar	87	700	400	.37	23
10. Douglas spruce	41	800	500	.51	32
11. White oak	218	2,200	1,000	.80	50
12. Overcup- "	216	1,900	1,000	.74	46
13. Post- "	49	3,000	1,100	.80	50
14. Cow- "	256	1,900	900	.74	46
15. Red "	57	2,300	1,100	.72	45
16. Texan "	117	2,000	900	.73	46
17. Yellow "	40	1,800	1,100	.72	45
18. Water- "	30	2,000	1,100	.73	46
19. Willow- "	153	1,600	900	.72	45
20. Spanish "	255	1,800	900	.73	46
21. Shagbark hickory	135	2,700	1,100	.81	51
22. Mockernut "	75	3,100	1,100	.85	53
23. Water- "	14	2,400	1,000	.73	46
24. Bitternut "	25	2,200	1,000	.77	48
25. Nutmeg- "	72	2,700	1,100	.78	49
26. Pecan "	37	2,800	1,200	.78	49
27. Pignut "	80	3,200	1,200	.89	56
28. White elm	18	1,200	800	.54	34
29. Cedar- "	44	2,100	1,300	.74	46
30. White ash	37	1,900	1,100	.62	39
31. Green "	10	1,700	1,000	.62	39
32. Sweet-gum	118	1,400	800	.59	37

*These results should be increased from 12 to 15 per cent to reduce them to 12 per cent moisture. See results corrected to 12 per cent moisture in table on p. 670.

3. *The Strength of Large Columns as compared with the Crushing Strength of Short Blocks.*—This investigation has not yet progressed far enough to enable a definite law to be announced, but they seem to justify the column formulæ given in Art. 446, p. 682.

4. *Variation of Strength with Position* across the section of a log, and also vertically in the trunk of the tree. These investigations are still incomplete.

5. *Relation of Strength to Moisture Condition.*—This was made the subject of a special investigation involving 1866 tests on small sticks two inches square. The laws derived from the larger tests were fully borne out. (Conclusion final.)

6. *The Uniformity of the Distribution of Moisture* in green and dry wood. The results showed that in sticks in which the moisture was evenly distributed when green it remained evenly distributed longitudinally while drying, the moisture percentages having been determined by taking full cross-sections $\frac{1}{4}$-inch thick entirely across the section. A difference was observed in the moisture determinations as between disks $\frac{1}{8}$ and $\frac{3}{8}$ inch thick, which the author attributes to the drying effect of the currents of air carried by the saw in cutting off the disks, this being relatively greater with the thinner sections. For this reason borings are better, but they cannot represent equally all parts of the cross-section as does the disk specimen. Disks cut with a rapidly moving circular cutting-off saw exhibited the effects of this drying action more than those cut with a hand-saw. There is always very much more moisture in green sap-wood than in green heart-wood.

7. *The Weakening Effect of Reabsorbed Moisture* is the same as that originally in the green timber. To determine this 224 tests of strength in compression endwise were made on sticks 2 inches square, one half of which were on the sap-wood and the remainder on the heart-wood of a single slab of short-leaf pine 4 inches thick cut especially for these tests and brought at once to the laboratory. Identical material (alternate sections) of sticks cut from this plank were used for the diminishing-moisture and for the increasing-moisture series, the reduction in moisture being carried to as near zero as was possible for testing in the open air, as the specimens reabsorb moisture very rapidly when removed from the dryer. Unfortunately the conditions of moisture were not as well distributed as was planned, but the results, as plotted in Fig. 601 for the sap-wood, are sufficient to show that *the weakening effect of a given percentage of moisture is the same whether this moisture be the original sap in the tree or whether it be reabsorbed water after the timber has been thoroughly seasoned or dried.* (Conclusion probably final.)

It also incidentally developed that *the maximum strength corresponds to about* 3 *or* 4 *per cent of moisture.* Since it is impossible to have wood, in any kind of service, at so low a moisture percentage (8 or 10 per cent being about a normal indoor minimum), this moisture condition of maximum strength has no economic significance.

8. *The Effect of Hot-air Drying on Strength.*—Two hundred tests were made to determine this, with the result that *for any temperature commonly used in drying lumber no detrimental effect on strength would be produced,*

aside from the checking action which might result from a too-rapid drying of the exterior portions of the sticks. (Conclusion probably final.)

9. *The Effect of Very High Temperatures and Pressures used in Drying* (as in the "vulcanizing" process).—In this investigation 210 mechanical tests were made on exactly similar material (long-leaf pine 4 inches square), a part of which had been submitted to the vulcanizing process in New York City, while the corresponding specimens had been dried by an air-blast at about 100° F.; many chemical analyses were also made to find if any chemical changes had occurred. *The results showed a slightly less strength for the "vulcanized" specimens, for like percentages of moisture,* with no appreciable chemical change. (Conclusion provisional.)

10. *The Effect of Long Immersion in Water on Strength.*—In this investigation 65 tests of strength have been made on material soaked in water for many months and the results compared with those on similar material (alternate sections of the same sticks), which passed through the regular tests. *So far as these tests go they indicate no loss of strength for six months' soaking in water.* (Conclusion provisional.)

Many other special investigations have been planned, but not yet executed for want of funds.*

443. Relations of Weight and Strength.†—"That within the same species the strength of wood varied with the dry weight (specific gravity), i.e., that the heavier stick is the stronger, has been known for some time. That this law of variation held good not only for a given species, but from species to species, in the pines of our Southern States, was indicated in Circular No. 12 of this Division. This fact becomes the more important in practical application, as the wood of these species of pines cannot as yet be disinguished at all by its anatomical structure, and only with difficulty and uncertainty by other appearances, while in the lumber market substitution is not infrequent. (See Art. 447.) It will, therefore, be best with these pines, when strength alone is desired, to inspect the material by finding the specific gravity (or weight per cubic foot), neglecting the species determination altogether.

"While this result of the exhaustive series of tests has been demonstrated for these pines and may be considered of great practical value, we can now extend the application of the law of relation between weight and strength a step farther, and state as an indication of our tests that probably *in woods of uniform structure strength increases with specific weight,* independently of species and genus distinction; *i.e., ceteris paribus, the heavier wood is the stronger.*

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* All experimental work stopped in March, 1896. It had been interrupted several times previously for lack of means. No special appropriation has ever been made for this work. It has all been done with small allotments made from the annual appropriations for the Forestry Division.

† Dr. Fernow, in Circular 15.

"We are at present inclined to state this important result with caution only as a probability or indication until either the test material and tests can be more closely scanned, or other more carefully planned and minutely executed series of detail tests can be carried on to confirm the truth of what the wholesale tests seem to have developed.

"In Figs. 603 and 604 the average strength of the different species in

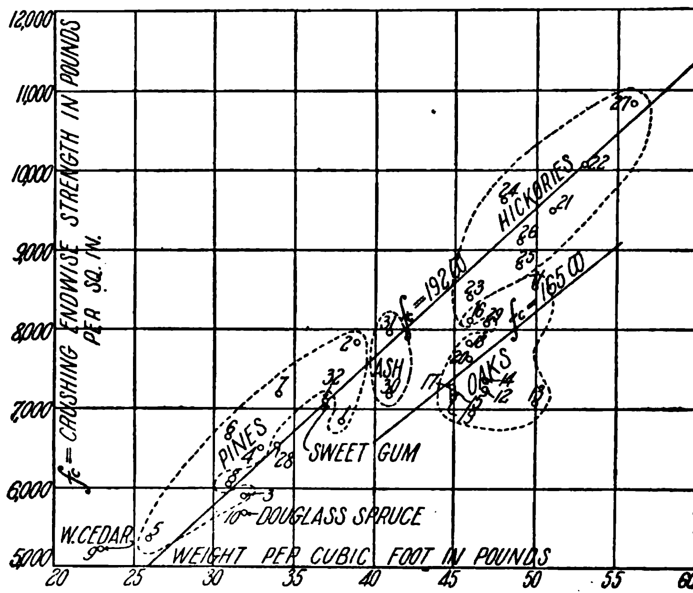


FIG. 603.—Showing the Relation between Weight and Crushing-endwise Strength of Various Timbers at the standard (12%) moisture, except the Southern pines, Nos. 1, 2, 3, and 4, which are plotted to 15 per cent moisture. (*U. S. Forestry Circular*, No. 15.)

compression endwise and in cross-bending, as found in Tables XLIX and LI, has been plotted with reference to the dry weight as given in Table XLVIII.*

"Considering that these tests and weight determinations (especially the latter) were not carried on with that exactness which would be required for a scientific demonstration of a natural law, that other influences, as cross-grain, unknown defects, and moisture conditions, may cloud the results, and that in the averaging of results undue consideration may have been given to weaker or stronger, heavier or lighter material, the relation is exhibited in spite of these wholesale methods with a remarkable degree of uniformity, bordering on demonstration.

"An exception is apparent in the oaks in that they do not exhibit the same relation of strength to weight shown to exist in the other species, and also there is a less definite law among the various species of oak when taken

* The results on the Southern pines, when reduced to 12 per cent moisture, as in the table on p. 670, fall from 600 to 1100 pounds higher in Fig. 603 and from 900 to 1700 pounds higher in Fig. 604.—J. B. J.

by themselves. The structure of oak wood being exceedingly complicated and essentially different from that of the wood of all other species under consideration (see Figs. 88 and 123), it may reasonably be expected that this species will not range itself with the others. In addition, the difficulty of

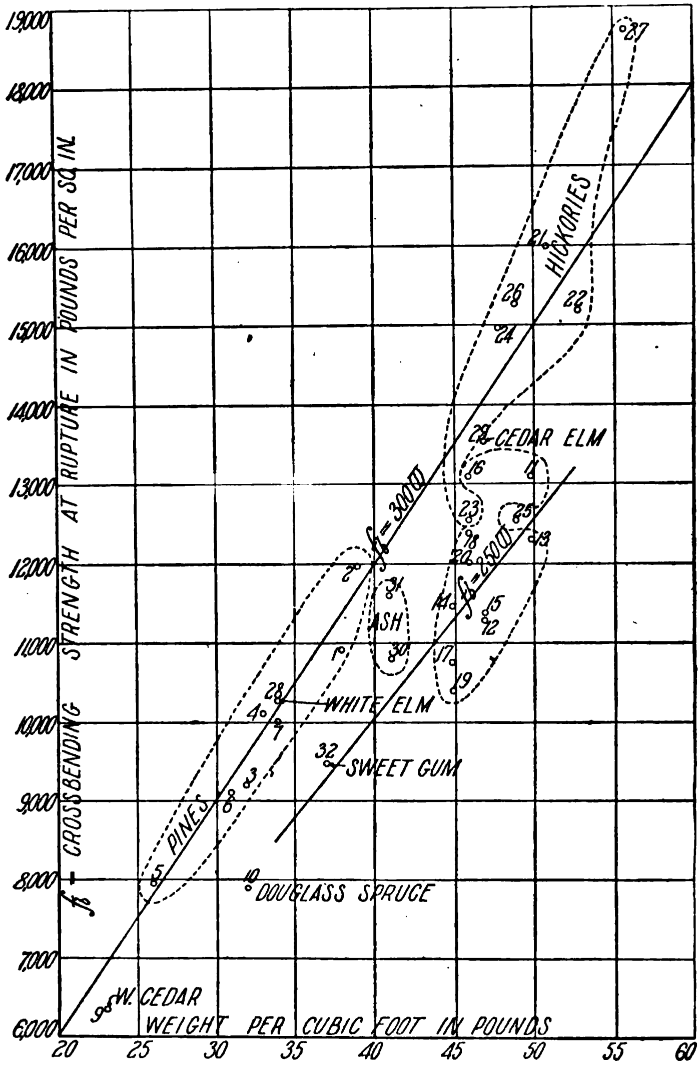


FIG. 604.—Showing the Relation between Weight and Cross-breaking Strength of Various Timbers at the Standard (12%) moisture, except the Southern pines, Nos. 1, 2, 3, and 4, which are plotted to 15 per cent moisture. (*U. S. Forestry Circular*, No. 15, 1897.)

seasoning oak without defects, or of even securing perfect material, may have influenced the results of tests so as to cloud the relationship within the genus.

"If further close study, supplemented by additional series of tests carefully devised to investigate this relationship, should uphold the truth of it, this result may be set down as the most important practical one that could be reached by these tests, for it would at once give into the hands of the wood-consumer a means of determining the relative value of his material as to strength and all allied properties by a simple process of weighing the dry material; of course with due regard to the other disturbing factors like cross-grain, defects, coarseness of grain, etc.

For instance, we would then have, from the results plotted in Figs. 603 and 604, approximately:

Crushing strength endwise of all timbers except the oaks, in pounds per square inch = 192 *times dry weight in pounds per cubic foot.* . (1)

Cross-breaking strength of all timbers except the oaks, in pounds per square inch = 300 *times dry weight in pounds per cubic foot.* . . . (2)

It thus appears that if the above law should be established it will only be necessary to determine the weight per cubic foot of any timber (perhaps not including the oaks), in order to be able to predict its strength, at least when used for beams and columns

444. The Factor of Safety to be used in timber-structure designing is now almost wholly a factor of ignorance, a large part of which ignorance it is the object of the U. S. Timber Tests to dispel, as is well stated by Dr. Fernow in Circular No. 15, as follows:

"As to factors of safety it may be proper to state that the final aims of the present investigations may be summed up into one proposition, namely, to establish *rational factors of safety.* It will be admitted by all engineers that the factors of safety as used at present can hardly be claimed to be more than guesswork. There is not an engineer who could give account as to the basis upon which *numerically* the factors of safety for wood have been established as '8 for steady stress, 10 for varying stress, 15 for shocks' (see Merriman's *Text-book on the Mechanics of Materials*); or as 4 to 5 for 'dead' load, and 5 to 10 for 'live' load (see Rankine's *Handbook of Civil Engineering*).

"The directions for using these indeterminate factors of safety, as given in the text-books, would imply that the student or engineer is, after all, relying on his judgment as to the modification he should make of such factors; that is, he is to add to this general guess his own particular guess. The factor of safety is, in the main, an expression of ignorance or lack of confidence in the reliability of values of strength upon which the designing proceeds, together with an absence of data upon which to inspect the material, and a provision for decay. With a larger number of well-conducted tests coupled with a knowledge of the quantitative as well as qualitative influences of various factors upon strength, and with definite data of inspection which allow ready sorting of material, the factor of safety, as far as it denotes the residuum of ignorance which may be assumed to remain, as to the character and behavior of the material, may be reduced to a minimum, restricting itself mainly to the consideration of the indeterminable variations in the actual and legitimate application of load, and to a provision for wear and decay.

"While the values given in these tables may claim to contain more elements of reliability than most of those published hitherto, much more work will have to be done before the above-stated aim will be satisfied."

445. Safe Loads for Rectangular Long-leaf Yellow-pine Beams.—Both Prof. Lanza's tests on wooden beams and those conducted by the author show that beams having a shorter length than twenty times the depth under a uniformly distributed load, or less than ten times the depth under concentrated loads, should be dimensioned for shearing lengthwise. As the total load which will cause the beam to shear is independent of the length, when shearing occurs before rupture in cross-breaking, the following table gives a constant load for all lengths shorter than the least length for rupture in cross-breaking. The table is based on a modulus of rupture of 1250 lbs. per square inch, or two thirds more than allowed for white-pine beams in Carnegie's Handbook. This is in accordance with the instructions there given. It gives a factor of safety of 5 on green beams and of 8 on dry beams, when known to be either long-leaf or Cuban yellow ("Georgia") pine. For white pine, cypress, and Oregon fir take 65 per cent of these

TABLE LIV. — SAFE UNIFORMLY DISTRIBUTED LOADS ON LONG-LEAF YELLOW-PINE BEAMS. BEAM ONE INCH THICK.

For other thicknesses, multiply the tabular values by the thickness of the beam.

Span in feet.	Kind of Stress.	Depth of Beam in Inches. 6	7	8	9	10	11	12	13	14	15	16	Kind of Stress.
5	Resistance to Shearing.	500	580	670	750	830	920	1000	1080	1170	1250	1330	Resistance to Shearing. Total Load.
6		500	580	670	750	830	920	1000	1080	1170	1250	1330	
7		500	580	670	750	830	920	1000	1080	1170	1250	1330	
8		500	580	670	750	830	920	1000	1080	1170	1250	1330	
9		500	580	670	750	830	920	1000	1080	1170	1250	1330	
10		500	580	670	750	830	920	1000	1080	1170	1250	1330	
11	Resistance to Cross-bending. Total Load.	450	580	670	750	830	920	1000	1080	1170	1250	1330	
12		420	570	670	750	830	920	1000	1080	1170	1250	1330	
13		380	520	670	750	830	920	1000	1080	1170	1250	1330	
14		350	480	630	750	830	920	1000	1080	1170	1250	1330	
15		330	450	600	750	830	920	1000	1080	1170	1250	1330	
16		320	425	550	710	830	920	1000	1080	1170	1250	1330	
17		300	400	520	670	820	920	1000	1080	1170	1250	1330	
18		280	375	480	630	770	920	1000	1080	1170	1250	1330	
19		270	350	470	600	730	880	1000	1080	1170	1250	1330	
20		250	330	450	570	690	850	1000	1080	1170	1250	1330	
21		240	320	425	540	650	810	950	1080	1170	1250	1330	
22		230	310	400	520	620	770	910	1070	1170	1250	1330	
23		220	300	380	490	600	730	970	1020	1170	1250	1330	
24		210	280	370	470	575	700	830	975	1140	1250	1330	
25		200	270	350	450	550	675	800	930	1100	1250	1330	
26		190	260	340	430	530	650	770	900	1060	1200	1330	
27		180	250	330	420	520	620	740	870	1020	1150	1320	
28		175	240	320	400	500	600	710	840	975	1110	1270	
29		175	230	300	380	480	580	680	820	930	1070	1230	

loads. For short-leaf yellow pine, Norway pine, spruce, oak, elm, and ash, take 80 per cent of these loads.

446. Strength of Wooden Columns.—A sufficient number of tests of columns has not as yet been made by the U. S. Forestry Divisionon which to base a general column formula. There was published, however, in the Report of Tests made at the Watertown Arsenal for 1882 a very complete series of tests of full-size columns, the average results of which are recorded in Tables LVI and LVII and plotted in Fig. 605. There were nearly 200 columns tested, part being white and part "yellow" pine. What particular species of yellow pine was used was not determined. Neither was the moisture condition of the timber examined. Judging from the results the timber must have been comparatively green. In a number of instances two or three sticks were bolted and keyed together, but in no instance did they

TABLE LV.—SAFE CONCENTRATED LOADS ON LONG-LEAF YELLOW-PINE BEAMS. BEAM ONE INCH THICK.

For other thicknesses, multiply the tabular values by the thickness of the beam.

Span in Feet.	Kind of Stress.	Depth of Beam in Inches.											Kind of Stress.
		6	7	8	9	10	11	12	13	14	15	16	
5	Resistance to Cross-bending. Total Load.	500	580	670	750	830	920	1000	1080	1170	1250	1330	Resistance to Shearing. Total Load.
6		430	570	670	750	830	920	1000	1080	1170	1250	1330	
7		370	480	630	750	830	920	1000	1080	1170	1250	1330	
8		320	425	560	700	830	920	1000	1080	1170	1250	1330	
9		280	380	490	675	775	920	1000	1080	1170	1250	1330	
10		250	340	440	560	690	840	1000	1080	1170	1250	1330	
11		225	310	410	510	630	770	910	1080	1170	1250	1330	
12		210	280	370	470	575	700	830	980	1170	1250	1330	
13		190	260	340	430	530	650	775	900	1050	1250	1330	
14		175	240	315	400	490	600	720	840	970	1110	1330	
15		165	225	300	375	470	560	670	780	910	1040	1180	Resistance to Cross-bending. Total Load.
16		160	210	275	355	430	525	625	730	850	980	1110	
17		150	200	260	335	410	495	590	690	800	920	1050	
18		140	190	240	315	385	470	560	650	760	870	990	
19		135	175	235	300	365	445	530	620	720	820	940	
20		125	165	225	285	345	425	500	590	680	780	890	
21		120	160	210	270	325	405	475	560	650	740	850	
22		115	155	200	260	310	385	455	535	620	710	810	
23		110	150	190	245	300	365	435	510	590	670	770	
24		105	140	185	235	290	350	415	485	570	650	740	
25		100	135	175	225	275	340	400	465	550	620	720	
26		95	130	170	215	265	325	385	450	520	600	680	
27		90	125	165	210	260	310	370	435	510	575	660	
28		90	120	160	200	250	300	355	420	485	555	685	
29		90	115	150	190	240	290	340	410	465	535	615	

* See Lanza's *Applied Mechanics*.

act as one solid stick would have done, but always as two or three single sticks would have acted if placed freely in the machine side by side, thus proving that *in all cases of composite wooden posts they must be treated as separate members, each taking its portion of the total load, and deflecting as though it stood alone.* A great deal of bad and even dangerous designing has resulted from violating this principle, and doubtless many fatal accidents have resulted from such a practice. It will be observed the composite members do not lie appreciably higher in Fig. 605 than the single sticks. Thus

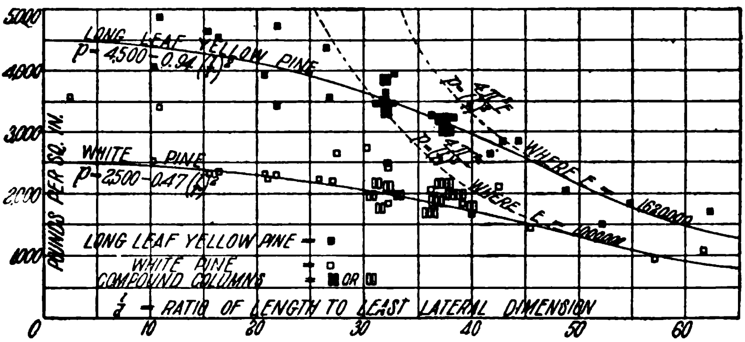


Fig. 605.—Tests of Full-size Comparatively Green Pine Columns made at the U. S. Watertown Arsenal. (See Report for 1882.) Each point plotted represents the average of three tests.

three sticks of yellow pine, 5.5 in. by 11.9 in. and 15 feet long, carried singly an average of 3470 lbs. per square inch. When exactly similar sticks were joined in pairs with packing-blocks and bolts, they carried in one test 3870 lbs. per square inch, and in another test 3530 lbs. per square inch; whereas if they had acted as one solid post they should have carried 4300 lbs. per square inch. When three sticks 4.8 in. by 11.5 in. by 15 feet long were packed and bolted side by side, they still deflected sideways, though 16.4 in. across now in this direction as against 11.5 in. in the other plane, and they carried 3110 lbs. per square inch in the one case and 3130 lbs. per square inch in the other. If they had acted as a single stick, they would have deflected in the other plane and at a load of 4300 lbs. per square inch. Similar results were obtained on white pine as shown in Table LVII and in Fig. 605.

These results also indicate that the author's parabolic column formula fit the experiments as well as any curve could, and hence he has drawn such curves in Fig. 605, and there given their equations, these being, for relatively green timber:

Ultimate strength for green yellow-pine columns,

$$p = 4500 - 1.0\left(\frac{l}{h}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Ultimate strength for green white-pine columns,

$$p = 2500 - 0.5\left(\frac{l}{h}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (2)$$

For dry timber these would become.

Ultimate strength for dry long-leaf pine columns,

$$p = 8000 - 2.0\left(\frac{l}{h}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (3)$$

Ultimate strength for dry white-pine columns,

$$p = 5000 - 1.0\left(\frac{l}{h}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (4)$$

If the same factors of safety be used here as were used in the tables of working loads on wooden beams, namely, 8 for dry and 5 for green timber, we would have:

Working load per square inch for long-leaf pine columns,

$$p = 1000 - \frac{1}{4}\left(\frac{l}{h}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (5)$$

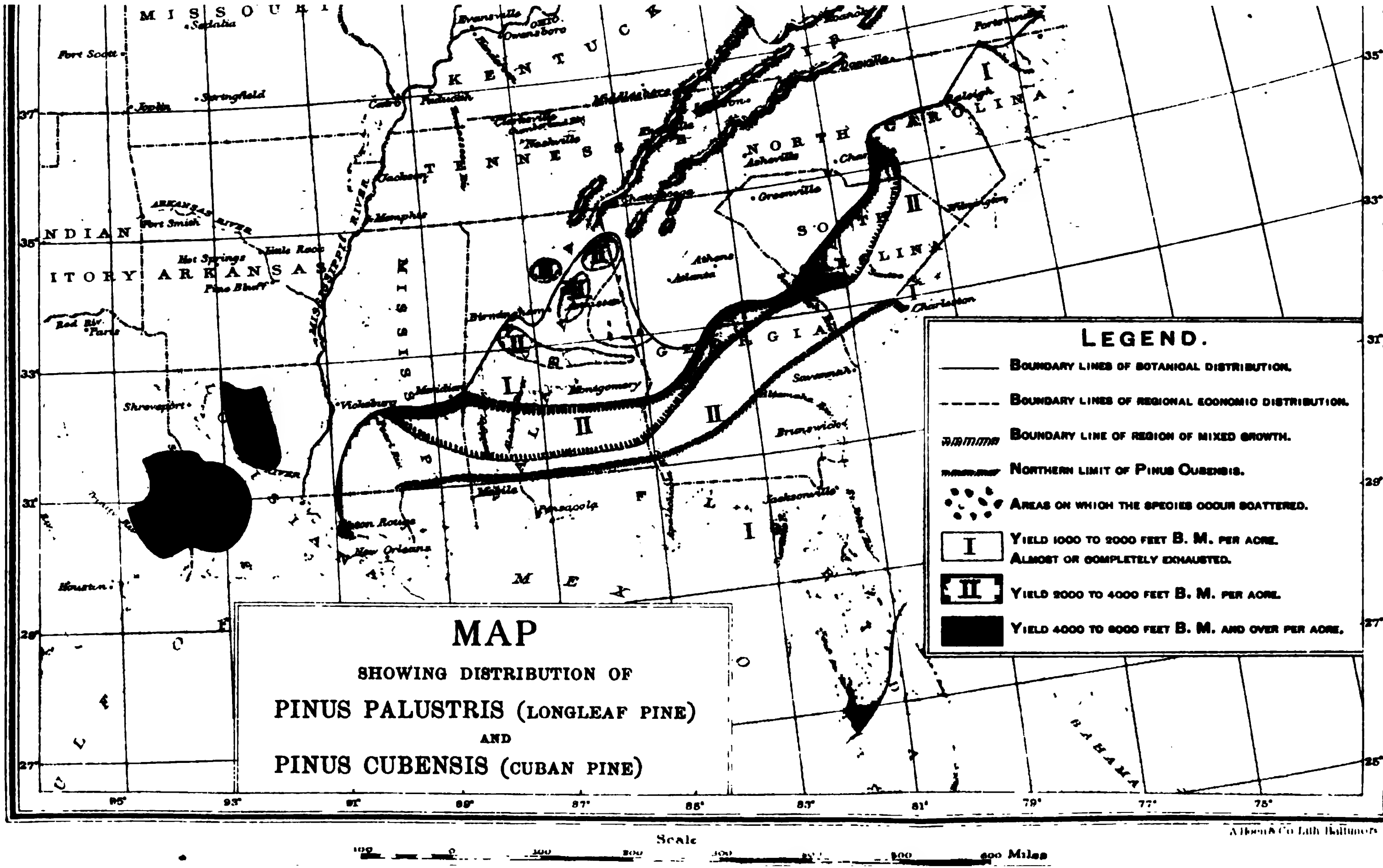
Working load per square inch for white-pine columns,

$$p = 600 - \frac{1}{8}\left(\frac{l}{h}\right)^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (6)$$

In all the above equations l = length of column having square ends, and h = least lateral dimension of the one or more single sticks of which the column is composed, both dimensions taken in the same unit of measure.

447. How to Distinguish Long-leaf from Short-leaf Pine Lumber.—The characteristic indications of these two species of pine become so merged that it is impossible to distinguish them when mixed in a consignment. If the short-leaf comes up to the long-leaf in specific gravity, in accordance with the law laid down in Art. 443, it would not be necessary to distinguish them, as they would then be of equal strength and value. As shown by Table XLVIII, the average weight per cubic foot of dry long-leaf pine is 38 lbs., while that of short-leaf pine is only 32 lbs. But as the lighter specimens of long-leaf may be no heavier than the heavier specimens of short-leaf, this is not an absolute guide.

The most nearly absolute criterion is the place of its growth. The long-leaf and short-leaf pines do not grow together to any great extent, as shown by Plates V and VI. These plates are reproduced from Forestry Bulletin No. 13 for the purpose of furnishing this particular criterion.


MAP
SHOWING DISTRIBUTION OF
PINUS PALUSTRIS (LONGLEAF PINE)
AND
PINUS CUBENSIS (CUBAN PINE)
LEGEND.
BOUNDARY LINES OF BOTANICAL DISTRIBUTION.
BOUNDARY LINES OF REGIONAL ECONOMIC DISTRIBUTION.
BOUNDARY LINE OF REGION OF MIXED GROWTH.
NORTHERN LIMIT OF PINUS CUBENSIS.
AREAS ON WHICH THE SPECIES OCCUR SCATTERED.
I YIELD 1000 TO 2000 FEET B. M. PER ACRE. ALMOST OR COMPLETELY EXHAUSTED.
II YIELD 2000 TO 4000 FEET B. M. PER ACRE.
YIELD 4000 TO 6000 FEET B. M. AND OVER PER ACRE.
MISSOURI
KENTUCKY
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
GEORGIA
ARKANSAS
MISSISSIPPI
FLORIDA
MEXICO
BAHAMA
Sedalia
Fort Scott
Springfield
Joplin
Evansville
Owensboro
Cairo
Paducah
Nashville
Memphis
Jackson
Fort Smith
ARKANSAS RIVER
MISSISSIPPI RIVER
Little Rock
Hot Springs
Pine Bluff
Red Riv.
Paris
Shreveport
Vicksburg
Meridian
Birmingham
Montgomery
Atlanta
Athens
Asheville
Greenville
Raleigh
Portsmouth
Wilmington
Charleston
Savannah
Brunswick
Jacksonville
Mobile
Pensacola
Baton Rouge
New Orleans
Houston
37°
35°
33°
31°
29°
27°
25°
95°
93°
91°
89°
87°
85°
83°
81°
79°
77°
75°
Scale
100
0
100
200
300
400
500
600 Miles
A. Hoen & Co. Lith. Baltimore

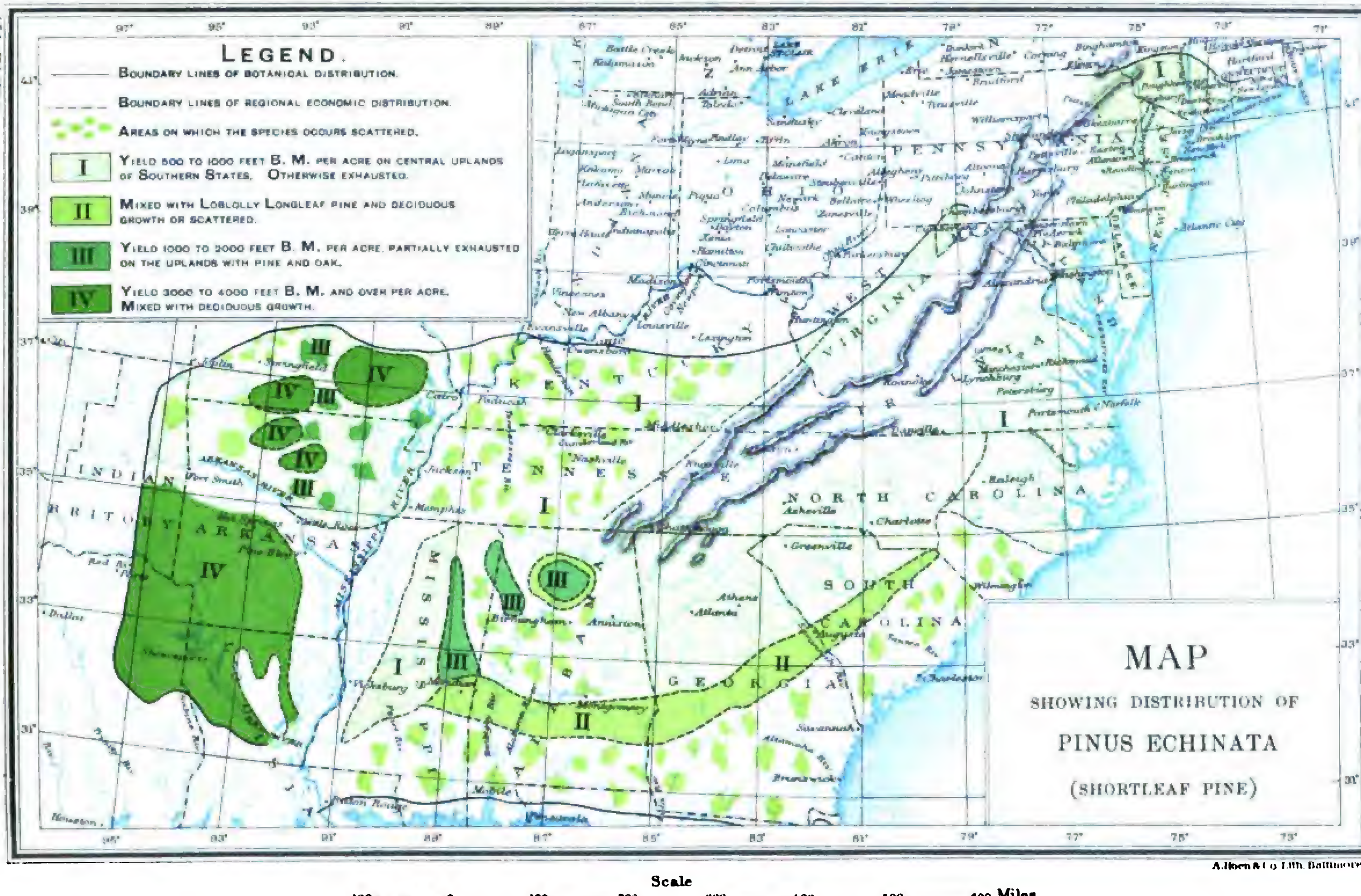

LEGEND.
Boundary lines of botanical distribution.
Boundary lines of regional economic distribution.
Areas on which the species occurs scattered.
I Yield 500 to 1000 feet B. M. per acre on central uplands of Southern States. Otherwise exhausted.
II Mixed with Loblolly Longleaf pine and deciduous growth or scattered.
III Yield 1000 to 2000 feet B. M. per acre, partially exhausted on the uplands with pine and oak.
IV Yield 3000 to 4000 feet B. M. and over per acre. Mixed with deciduous growth.
MAP
SHOWING DISTRIBUTION OF
PINUS ECHINATA
(SHORTLEAF PINE)
PENNSYLVANIA
OHIO
KENTUCKY
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
GEORGIA
ALABAMA
MISSISSIPPI
ARKANSAS
INDIAN TERRITORY
WEST VIRGINIA
VIRGINIA
LAKE ERIE
MISSISSIPPI RIVER
Scale
100 0 100 200 300 400 500 600 Miles
A. Hoen & Co. Lith. Baltimore

TABLE LVI.—COMPRESSIVE STRENGTH OF UNSEASONED YELLOW-PINE COLUMNS.

(From *Rep. Wat. Ars. Tests*, 1882.)

Number of Tests Averaged.	Length of Column in Inches.	Least Lateral Dimension in Inches.	Greatest Lateral Dimension in Inches.	Ultimate Strength in Pounds per Square Inch.	Ratio of Length to Radius of Gyration.	Ratio of Length to Least Lateral Dimension	Remarks.
	l	d	h	p	$\frac{l}{r}$	$\frac{l}{d}$	
4	60	5.48	5.51	4 868	38	11	
3	90	5.46	5.58	4,537	57	16	
3	120	5.48	5.50	4,738	76	22	
3	150	5.50	5.51	5,077	95	27	
3	180	5.48	5.50	3,962	114	33	
3	210	5.48	5.48	3,242	133	38	
2	240	5.42	5.46	2,868	154	44	
3	270	5.55	5.57	2,064	169	49	
3	300	5.46	5.58	1,856	190	55	
3	330	5.30	5.31	1,709	216	62	
3	80	7.76	9.78	4,085	36	10	
3	120	7.76	9 74	4,603	54	16	
3	160	7.73	9.74	3,935	72	21	
3	200	7.56	9.65	4,384	92	27	
3	240	7.59	9.68	3,494	108	31	
3	280	7.69	9.75	3,300	126	36	
3	320	7.44	9.28	2,873	149	43	
3	180	5.63	15.6	3,658	111	32	
4	180	6.70	15.6	3,594	93	27	
3	180	8.24	16.3	3,445	76	22	
3	180	4.31	11.5	2,663	145	42	Sticks like those joined together
3	180	5.52	11.9	3,472	113	33	" " " " "
3	180	5.62	11.7	3,869	111	32	2 sticks with 3 packing-blocks
3	180	5.60	11.7	3,530	111	32	2 sticks with packing-blocks at the ¼ points
3	180	5.62	11.6	3,865	111	32	2 sticks, keyed, with uneven bearings
3	180	4.80	11.5	3,110	130	37	3 sticks with 3 packing-blocks
3	180	4.86	11.5	3,130	128	37	Same, but swelled ¾ in. at centre

By demanding the way bills on all consignments coming directly from the mills to fill any particular order (which is now the almost universal custom), one may learn the exact locality of the timber's growth, and by reference to Plates V and VI a very high degree of probability can be established as to the species. As a rule the long-leaf pine is of much slower growth than the short-leaf, and hence its annual rings are much narrower. It also contains more rosin. Very fair characteristic views of the standing timber of these two species may be had from Figs. 606 and 607. The needles of

FIG. 606.—Specimens of Long-leaf Pine-trees growing in Open Woods

FIG. 607.—Specimen of a Short-leaf Pine-tree growing in the Open. (Taken from *U. S. Forestry Bulletin* No. 13.)

TABLE LVII.—COMPRESSIVE STRENGTH OF UNSEASONED WHITE-PINE COLUMNS.

(From *Rep. Wat. Ars. Tests*, 1882.)

Number of Tests Averaged.	Length of Column in Inches.	Least Lateral Dimension in Inches.	Greatest Lateral Dimension in Inches.	Ultimate Strength in Pounds per Square Inch.	Ratio of Length to Radius of Gyration.	Ratio of Length to Least Lateral Dimension	Remarks.
	l	d	h	p	$\frac{l}{r}$	$\frac{l}{d}$	
1	15	5.50	5.50	3,570	9.4	3	These sticks were probably dryer than the longer columns
2	60	5.48	5.48	3,400	37.9	11	
3	90	5.50	5.50	2,357	56.7	16	
3	120	5.46	5.46	2,299	76.1	22	
3	150	5.50	5.48	2,643	94.8	27	
3	180	5.43	5.43	2,744	115	33	
3	210	5.36	5.36	1,841	136	39	
3	240	5.27	5.28	1,455	158	46	
3	270	5.18	5.19	1,501	181	52	
3	300	5.25	5.25	952	198	57	
3	330	5.34	5.35	1,080	214	62	
3	80	7.73	9.66	2,527	35.9	10	
3	120	7.73	9.70	2,334	53.8	16	
3	160	7.66	9.58	2,307	72.4	21	
3	200	7.75	9.65	2,225	89.4	26	
3	240	7.45	9.40	2,445	112	32	
3	280	7.70	9.62	2,072	126	36	
3	320	7.47	9.36	2,113	148	43	
3	180	5.60	15.6	1,874	111	32	
3	180	6.60	15.6	2,204	95	27	
3	180	8.48	16.5	2,222	74	21	
3	180	4.45	11.6	1,672	139	40	Sticks like those joined together
3	180	5.55	11.6	2,432	112	32	" " " " "
3	180	4.50	11.6	1,792	139	40	2 sticks with bolts and packing-blocks
3	180	5.60	11.6	1,880	111	32	2 " " " " " "
3	180	5.60	11.7	1,991	111	32	2 sticks swelled ¾ in. at centre
3	180	5.60	11.6	1,947	111	32	2 sticks and 3 keys, bolted
3	180	5.60	11.7	1,974	111	32	2 sticks keyed at the ¼ points
3	180	5.60	11.7	2,102	111	32	2 sticks keyed at ends, packed at centre, but with uneven bearings
3	180	4.98	11.7	1,746	125	..	3 sticks with 3 packing blocks
3	180	4.86	11.6	1,913	128	..	Same, but swelled ¾ in. at centre
3	180	4.68	11.7	1,950	133	..	4 sticks with 3 packing-blocks
3	180	4.88	11.6	1,998	128	..	Same, but swelled ¾ in. at centre

the long-leaf pine are some 12 inches long, while those of the short-leaf pine are only about 2 inches long.

448. The Strength of Bamboo is very great for [illegible] weight, as shown by Table LVIII. Thus, taking 17,300 lbs. per square inch as the apparent

TABLE LVIII.—STRENGTH OF BAMBOO IN CROSS-BENDING.

(Tests made by the author.)

Outside Diameter between Joints in Inches.	Inside Diameter between Joints in Inches.	Area of Cross-section between Joints in Square Inches.	Length between Supports in Inches.	Number of Joint-lengths between Supports.	Weight of Specimen between Supports in Pounds.	Modulus of Elasticity in Pounds per Square Inch.	Modulus of Rupture in Cross-bending.	Modulus of Strength at the Apparent Elastic Limit.	Ultimate Deflection of Specimen in Inches.	Deflection at the Apparent Elastic Limit in Inches.	Elastic Resilience in Inch-pounds per Pound Weight of Specimen.
1.25	0.91	......	24	3	0.539	2,230,000	19,600	13,000	1.1	0.54	156
1.25	.93	......	28	3	.578	2,200,000	23,200	15,800	3.0	0.89	249
1.16	.86	......	28	3	.516	2,510,000	25,000	16,400	2.0	0.79	196
1.04	.78	......	24	3	.375	2,500,000	25,800	15,900	2.2	0.65	182
0.87	.63	.....	22.5	3	.266	2,500,000	25,800	17,200	2.0	0.73	205
0.71	.51	......	25	3	.203	3,020,000	27,600	17,200	2.3	0.90	162
0.40	.24	......	7.5	1	.053	2,100,000	41,100	23,300	1.1	0.28	337
0.54	.38	......	8.0	1	.029	1,960,000	30,900	19,700	0.65	0.21	245
			Mean Values........			2,380,000	27,400	17,300	....		216

elastic limit strength per square inch of bamboo in cross-breaking (using the formula $M = \frac{fI}{y_1}$, and computing I for the actual annular section), we find, by comparing with the results in Table XLVIII, that the strongest timber there listed, namely, pignut hickory, is far below it in strength, having a modulus at this limit of only 12,600 lbs. If we compare the bamboo weight for weight with this, the strongest timber found in the Forestry Division tests, to give a certain cross-breaking strength on a given span, as for instance 28 inches, and taking the timber in the form of a solid rectangular cross-section, we find that to carry a load of 440 lbs. at the centre, which was carried by the second specimen in Table LVIII, it would require a stick 1.14 in. square in cross-section. This would weigh 1.4 lbs., whereas the bamboo specimen weighed only 0.58 lbs. That is to say, *bamboo is just twice as strong as the strongest wood in cross-bending, weight for weight, when the wood is taken in specimens with a square and solid cross-section.* The same holds true also for crushing endwise.

449. The Holding Force of Nails.—One of the most valuable properties of wood is the facility with which boards may be attached by means of nails, and the strength of such attachments. The holding force of nails and spikes in different woods is therefore of considerable importance. In Fig. 608 the starting resistances against the drawing out from dry oak wood, of nails having different styles of points, are shown graphically. The cut nails exhibit a much greater holding force than do the wire nails, and a slightly sharpened

point gives the highest resistance for each species. This figure exhibits the holding force of different nails, per square inch of embedded surface, when

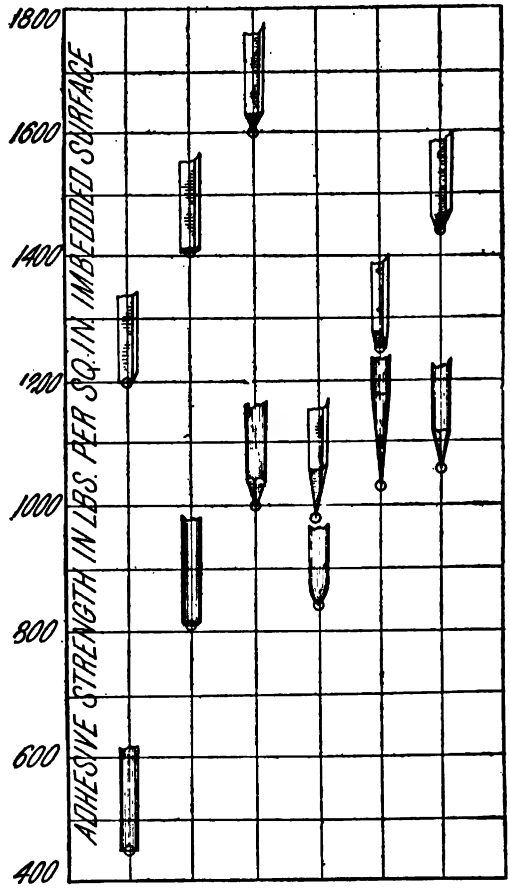


FIG. 608.—Relative Adhesive Strength of Wire and Cut Nails (in Oak Wood) as affected by the shapes of their points. (*Engr. News*, vol. xxxi. p. 24.)

driven laterally into dry oak wood. Evidently for the softer woods the resistance to drawing is very much less, and so is the resistance when driven endwise into the stick.

CHAPTER XXXIII.

STRENGTH OF IRON AND STEEL WIRE, AND WIRE ROPE.

450. The Strength of Wire increases with repeated drawings, as indicated in Fig. 609. As the strength increases the ductility decreases. By anneal-

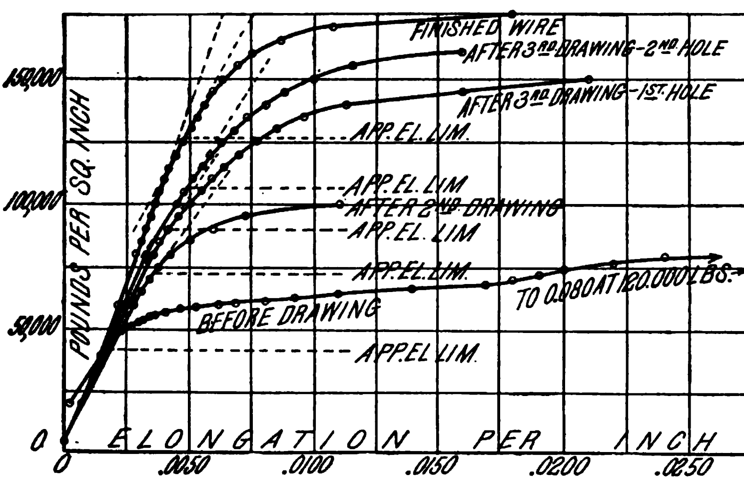


Fig. 609.—Showing Increase in Strength in Drawing Steel Wire three times from 0.216 in. to 0.10 in. diameter. (*Rep. Wat. Ars.*, 1890.)

ing, the ductility is restored and the strength again reduced preparatory to further drawing. The final product is given such a temper as its particular use demands.

The increase in the strength of wrought iron with rolling to small rods and then drawing through dies is shown in Fig. 610, where the diameters vary from 0.8 inch in rolled rods to 0.001 inch in fine wires, the tensile strength increasing from 50,000 lbs. to 110,000 lbs. per square inch.

Fig. 249, p. 309, contains stress-diagrams of steel piano-wires having a tensile strength of about 350,000 lbs. per square inch. In the same series of tests other wires, about 0.03 inch in diameter, showed a tensile strength as high as 447,000 lbs. per square inch.* These wires were of high-carbon steel, practically free from sulphur and phosphorus, the chemical composi-

* *Rep. Wat. Ars. Tests*, 1894, p. 347

tion of the strongest wires being: combined carbon, 0.80; manganese, 0.17; silica, 0.41; sulphur, 0.015; phosphorus, 0.020.

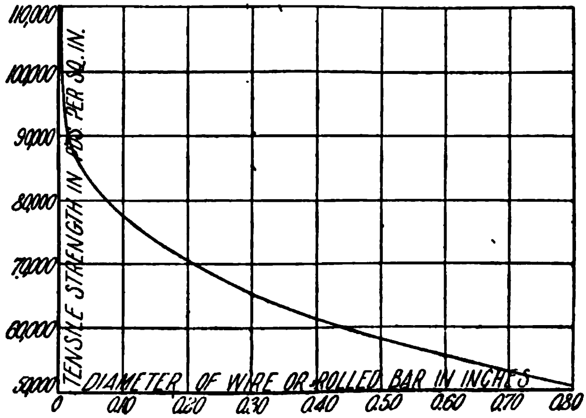


FIG. 610.

The modulus of elasticity of these wires was 28,400,000, showing that throughout the entire range of tensile strength of steel from 50,000 to

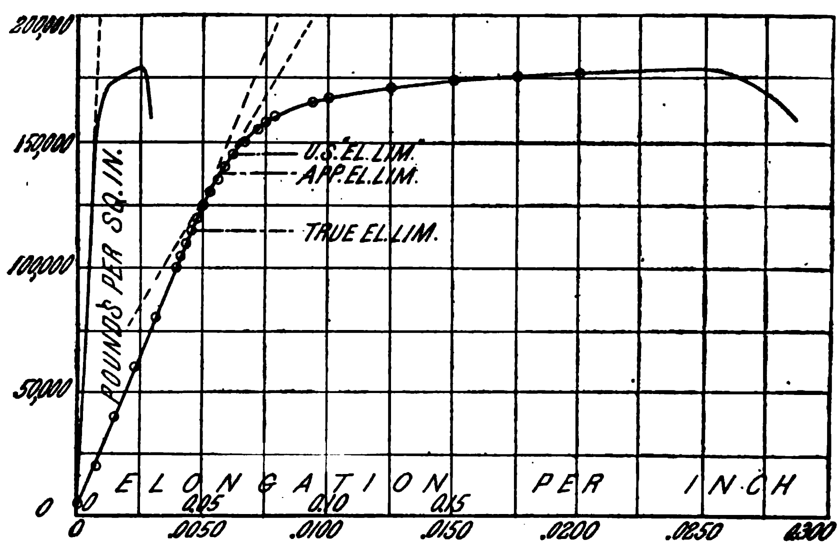


FIG. 611.—Average of Nine Tests in Tension on Steel Wires, showing Relation of Elastic Limits. (*Wat. Ars. Rep.*, 1890.)

450,000 lbs. per square inch the ratio of stress to elastic deformation is practically constant.

This material has no "yield-point," such as is always found with the low-carbon steels, as appears from the diagrams in Fig. 611. This diagram

exhibits the advantage of adopting an arbitrary "apparent elastic limit," as described in Art. 13, p. 18. As here shown (Fig. 611) this apparent elastic limit is well above the true elastic limit, but well below the so-called "elastic limit" as given in the original published report of this test. It corresponds to a permanent set of less than 0.0004 of the length of the specimen, which would be quite imperceptible and hence of no significance. The total stretch of the specimen at rupture is only 2.8 per cent, or about two per cent if measured after rupture. This is the quality of wire commonly employed in the manufacture of high-grade wire rope for power transmission, cable railways, and the like. Three per cent elongation, measured after rupture, is very large for this quality of material.

Mr. J. Bucknall Smith gives the following average values of the strength of iron and steel wires *:

	Lbs. per Sq. In.
Bright hard-drawn iron wire....	80,000
Bessemer steel wire..............................	90,000
Mild open-hearth steel wire	130,000
High-carbon open-hearth steel wire.........	180,000
Crucible cast steel wire (patent tempering).........	220,000
Crucible cast steel (plough † quality).....	240,000

"Bright wire" is that which remains untreated after the last drawing. If it is annealed or tempered in any way after the last drawing, it is left black.

451. The Strength of Steel-wire Rope is difficult to obtain from short samples because of the small stretch of the wires, and the fact that some of them are more rigidly held than others. In order to grip and hold these ends with equal effectiveness various devices have been tried, two of the most successful of which are here described.

The first method is to grip the rope as a whole, without uncoiling the ends, by means of grooved wedges moving in a steel-plate holder as shown in Fig. 612. This has worked successfully and requires no preparation of the specimen.

The author has used cast-iron and steel holders having conical openings for receiving the prepared ends of the cable as shown in Fig. 613. Before cutting off the sample it should first be bound tightly with soft wire, some six inches from the ends, and then cut off. The intervening length of specimen should be wrapped tightly with tarred cord to hold the strands to their true position. The ends are then inserted in the sockets, the strands opened up, and each individual wire turned back upon itself as shown at the

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* In *Mining Journal*, June 6—July 11, 1896.

† So called because it was first used for drawing machine-ploughs in England; hence it is now known as "plough-steel."

right of the figure. The ends are then boiled in caustic soda to remove all grease, thoroughly washed in hot water, and then dipped in chloride of

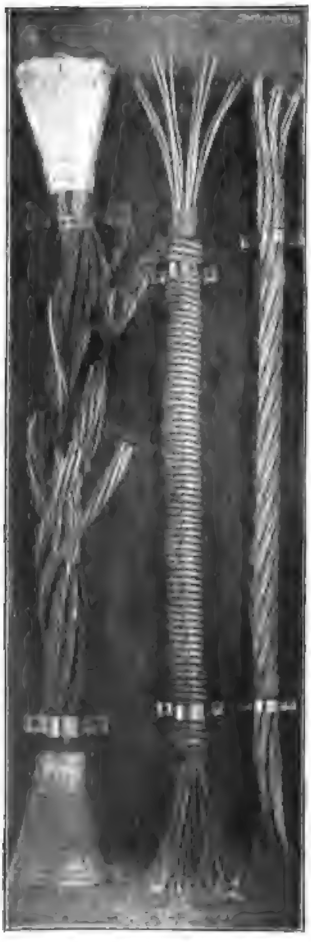


FIG. 613.—Wire Cables prepared for Testing, as practised at Yorkshire College, Eng., and described in *Engineering*, Sept. 11, 1896.

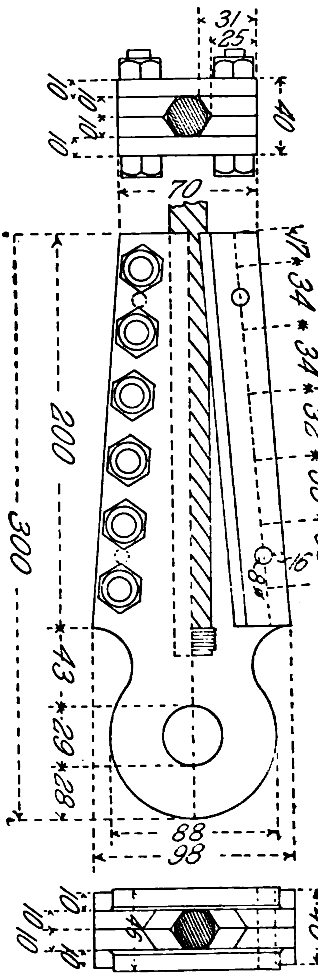


FIG. 612.—Wedge-grips for holding Wire Rope for Testing. All dimensions are given in millimeters. (*Berlin Testg. Lab. Communications*, 1893.)

zinc and afterwards in molten solder, thus tinning over each bent wire. The ends are then drawn into the conical dies and an alloy of lead, tin, and antimony cast around it.* The specimens are now ready to go to the testing machine, where the pulling force is applied to the conical sockets in some suitable manner. Split steel sockets, bolted together, may be used if preferred. By this means about 95 per cent of the combined strength of

* Tetmajer uses 8 parts tin, 1 part copper, and 1 part antimony for iron and mild-steel wires, while for hard-steel wires of great strength he uses 9 parts lead, 2 parts antimony, and 1 part bismuth.

the individual wires can be developed in the rope.* If the wedge grips can be made to give satisfaction, however, they are much to be preferred.

In long wire ropes on a straight pull the strength of the rope may be taken as about equal to the average strength of the individual wires if these are all of about the same ductility and ultimate strength. If the wires differ greatly in ductility, the ultimate strength of the rope is the average resistance of the wires at that percentage of elongation which corresponds to the total elongation of the least ductile samples. It is common to assume the rope to have 85 per cent of the total strength of the wires when tested individually.

Wire-rope pulleys, sheaves, and barrels should have a diameter not less than thirty times the circumference (or say one hundred times the diameter) of the ropes running upon them, to prevent excessive bending strains in the ropes.

In Table LIX are given a summary of several hundred tests of high-grade steel wire and of the ropes it was made up into. As these tests were conducted by Prof. Tetmajer with great care, they can be relied on as giving the facts for this class of rope. The material found in these specimens, which were all taken from ropes actually in service in Switzerland, is superior to that usually found serving similar purposes in America. They may be

TABLE LIX.—RÉSUMÉ OF TESTS ON CRUCIBLE CAST-STEEL WIRE AND WIRE ROPE USED ON CABLE RAILWAYS IN SWITZERLAND.

(From *Tetmajer's Communications*, vol. IV. p. 272.)

Number of Cable.	Test of Entire Cable. (Each the Mean of Two Tests.)			Test of Individual Wires. (Each the Mean of Eleven Tests.)					Ratio of Strength of Cable to Average Strength of Wires.
				Tension Test.			Torsion.	Bending.	
	Diameter in Inches.	Tensile Strength in Pounds per Square Inch of Actual Wire Section.	Per Cent of Elongation in 8 ft.	Strength in Pounds per Square Inch.	Per Cent of Elongation in 20 Inches at Rupture.	Work of Deformation in Foot-pounds per Cubic Inch.	Number of Twists (of 360°) in 8 Inches.	Number of Bends of 180° Each on ⅛-in. Radius.	
1	1.65	220,000	3.12	265,000	3.4	6,400	27.5	11.4	83
2	1.67	117,000	7.45	122,000	9.4	9,500	61.5	11.8	96
3	1.18	205,000	2.61	213,000	3.0	4,600	35.1	17.8	96
4	1.43	191,000	3.30	207,000	3.4	5,000	44.5	18.0	93
5	1.00	184,000	3.92	191,000	3.85	5.300	...	15.1	96
6	1.00	184,000	3.28	190,000	4.0	5,700	52.6	14.8	97
7	1.38	180,000	2.37	222,000	3.0	4,600	33.7	11.0	77
8	1.30	226,000	3.00	247,000	3.3	5,700	21.7	9.6	92
9	1.26	210,000	3.15	238,000	3.3	5,400	31.1	9.4	89
10	1.00	190,000	2.40	190,000	2.7	3,400	48.1	18.8	100
Averages (omitting No. 2).		199,000	2.98	217,000	3.29	5,100	36.8	14.0	92

* See article in *Engineering*, Sept. 11, 1896.

regarded, therefore, as setting a pattern for American manufactureres to strive to attain to.*

It is the opinion of the author of this work, who has had considerable experience in testing high-grade steel wire, that the tension test, taken so as to furnish a complete stress-diagram (as Tetmajer took his), furnishes about all the information required. The percentage of elongation is the best indication of ductility, or pliability, and the area of the stress-diagram, reduced to foot-pounds of work done per cubic inch of metal as in Table LIX, gives the best indication of the value of the wire where a very high tensile strength must be combined with as great a toughness as possible. Next to this comes the cold-bending test. The torsion test is, in the opinion of the author, of doubtful value, except that it may serve to indicate the *uniformity* of the material by testing many samples. The most significant result, as indicating wearing quality or long life, is the percentage of elongation in the tension test. That an average strength of individual wires of 217,000 lbs. per square inch should be coupled with an average elongation of 3.29 per cent would mark an unusually happy combination of strength and toughness, were it not for the fact that this elongation was automatically recorded, and that it was *at* rupture and not *after* rupture. The elastic stretch at 217,000 lbs. per square inch would be about 0.8 of one per cent, so that if this be subtracted we have only 2.5 per cent elongation if measured *after* rupture. This is, however, a high average elongation for such great strength.

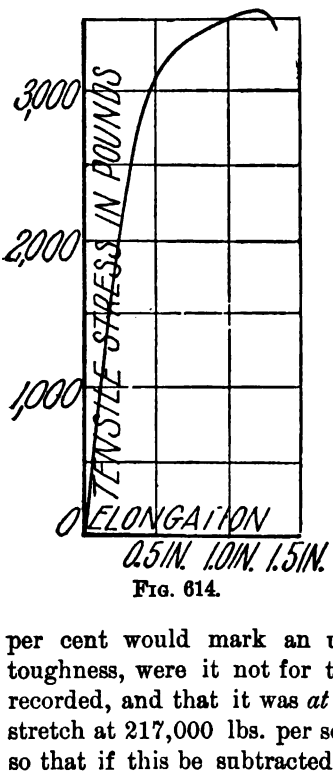


FIG. 614.

Fig. 614 shows an autographic stress-diagram of a high-grade steel wire taken by the author on an English wire 0.15 in. diameter, on the machine shown in Fig. 615. The diagram gives the following results:

Ultimate strength in pounds per square inch..........	=	200,000
Percentage of elongation in 48 inches................	=	2.5
Work of deformation in inch-pounds per cubic inch....	=	4,000
Number of twists in 8 in. (torsion test)..............	=	21
Number of bends of 180° each on $\frac{1}{8}$ in. radius (bending test)..	=	3

* All the instruments used, the methods employed, and the results obtained are given in great detail in the original volume. This volume (IV) can now be had only in a French translation.

By comparison with Table LIX it is seen that this wire is very inferior to those there recorded.

The tension test was made on the machine shown in Fig. 615, while the bending test was made on the machine shown in Fig. 617.

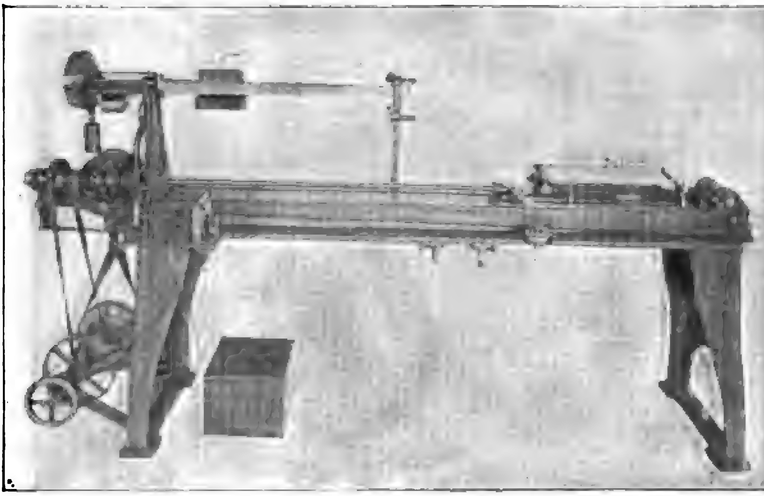


FIG. 615.

452. Shop Tests of Wire.—The most significant shop tests on wire are:

1. Tension tests with autographic stress-diagrams.

2. Cold-bending tests, through 180°, back and forth about jaws having a radius of $\frac{1}{8}$ inch, or equal to the diameter of the wire.

3. Torsion tests on a length of 8 inches with self-recording attachment, giving number of revolutions.

The instruments shown in Fig. 615 or 616 are very satisfactory for making the tension tests. Both give the record complete after the specimen is placed and the machine started, without any personal attention whatever. In Fig. 615 the poise is operated electrically, while in Fig. 616 the load is indicated by the deformation of the heavy spring at the top. This gives also the downward dip of the diagram at rupture without any special appliances, while in Fig. 616 this is also done by stopping the test and crossing a band so as to move the poise backward.*

The cold-bending machine shown in Fig. 617 is a very satisfactory one. By having a number of pairs of jaws with different radii these may always be made about equal to the diameter of the wire. A schedule may then be prepared for the workman, instructing him to use certain numbers of jaws with given numbers of wires.

The torsion tests are made on a machine like that shown in Fig. 319, p. 392. This, however, has no revolution-counter attachment shown

* Some improvements have now been introduced in this machine by the maker.

Other tests are sometimes resorted to to determine wearing quality. Thus Mr. J B. Stone, C.E.,* has arranged a series of small pulleys (about

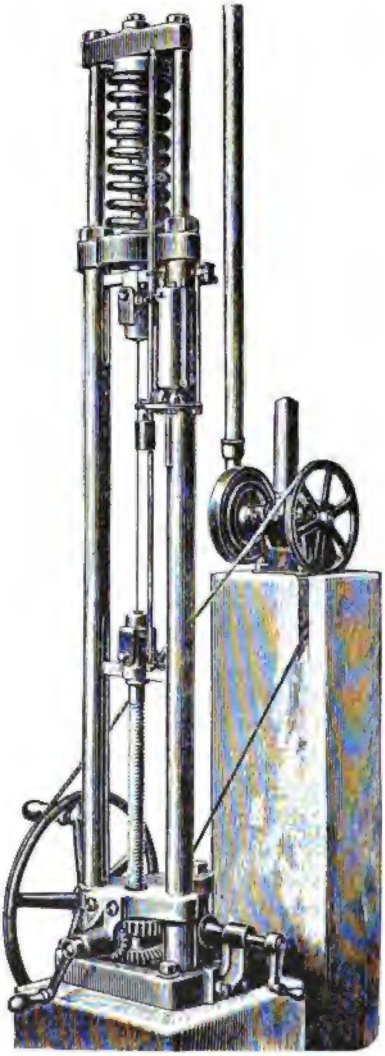


FIG. 616.—The Amsler-Laffon & Son Wire-testing Machine, used by Prof. Tetmajer. (See his *Communications*, vol. IV. p. 239.)

6 or 8 inches in diameter), in such relative positions that a wire drawn over them is bent alternately in opposite directions. A given tension is then put on a loop of wire, and it is run over this series, which is provided with a

* Of the Washburn Moen Works, at Worcester, Mass.

revolution-counter, until it breaks. The counter then gives the running record of the wire. Mr. Stone then takes the product of the number of revolutions into the tensile strength and calls this the "hoisting value" of

FIG. 617.—Wire Setting for Cold-bending Test.

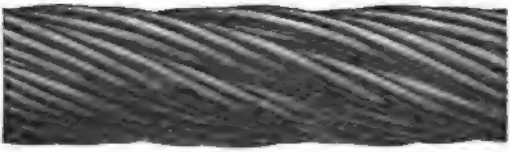


FIG. 618.—Lang-lay Wire Rope, new.

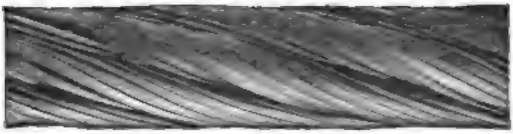


FIG. 619.—Lang-lay Wire Rope, well worn.

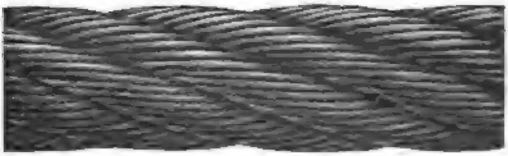


FIG. 620.—Ordinary Lay, new.

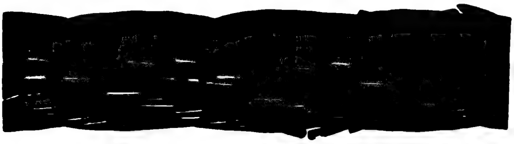


FIG. 621.—Ordinary Lay, well worn.

the wire. For comparing wires of the same diameter he finds by experience that this is a good measure of their true worth in service when made into ropes, and his experience with this test dates back to 1882, when he first

TABLE LX.—BREAKING-LOADS AND EQUIVALENT SIZES AND WEIGHTS OF WIRE ROPES.

(From J. Bucknall Smith's Articles on Wire Rope in *Mining Journal*, June 6 to July 11, 1896.)

Sizes of Ropes, and Approximate Weights per Fathom.			Calculated Breaking Load of Ropes.	Circumference of Rope in Inches.					
Circumference of Rope in Inches.	Weight of Rope made entirely of Wire, per Fathom.	Weight of Rope made with Hemp Centre Core, per Fathom.	Breaking Strength about	Best Selected "Extra Plough"-steel Wire.	Best Selected "Plough"-steel Wire.	Best Selected Improved Patent Crucible-steel Wire.	Patent Crucible-steel Wire.	Best Selected Bessemer-steel Wire.	Best Selected Charcoal-iron Wire.
Compound Strands	Pounds, about		Net Tons.	Compound Strands					
...	....	..	168	6¼	6¾	...	...	...	...
6¾	48	42	156	6	6½	...	...	...	...
6½	44	38	143	5¾	6¼	6¾	...	...	...
6¼	40	35	132	5½	6	6½	...	...	...
6	37	32	123	5¼	5¾	6¼	...	...	...
5¾	34	29	112	5	5½	6	...	...	...
5½	30	26	104	4¾	5¼	5¾	6¾	...	...
5¼	27	24	98	...	5	...	6½	...	...
5	25½	23	95	4½	...	5½	...	...	...
4¾	24	22	90	...	4¾	5¼	6¼	...	...
4½	22	20	84	4¼	...	5	6	6¾	...
4¼	19½	17½	78	...	4½	...	5¾	6½	...
4	18½	16	73	4	4¼	4¾	5½	6¼	...
...	....	....	67	...	...	...	5¼	6	...
...	....	....	65	...	4	4½	...	...	6¾
3⅞	18¼	15¾	62	3¾	...	4¼	5	5¾	6½
3¾	17	14½	58	3⅝	3⅞	...	...	...	...
3⅝	15½	13½	56	...	...	4	4¾	5½	6¼
3½	13½	11¾	54	3½	3¾	...	...	5¼	...
3⅜	12	10½	50	3⅜	3⅝	...	4½	5	6
3¼	10¾	9	46	3¼	3½	3⅞	4¼	4¾	5¾
3⅛	10	8½	43	3⅛	3⅜	3¾	...	...	5½
3	9½	8	40	...	3¼	3⅝	4	4½	5¼
2⅞	8½	7½	38	3	...	...	...	...	...
2¾	8	7	37	...	3⅛	3½	...	4¼	5
2⅝	7	6¼	36	...	...	3⅜	3⅞	...	4¾
2½	6	5½	35	2⅞	...	...	...	...	...
2⅜	5½	5 1/10	34	...	3	3¼	3¾	...	...
2¼	4 8/10	4¼	32	2¾	...	...	...	4	4½
2⅛	4 3/10	4	30	...	2⅞	3⅛	3⅝	...	...
2	4 1/5	3 8/10	28	2⅝	...	3	3½	3⅞	4¼
1⅞	4	3½	26	2½	2¾	2⅞	3⅜	3¾	4
1¾	3 2/5	3 1/10	25	...	2⅝	...	3¼	3⅝	...
1⅝	....	2¾	24	2⅜	...	...	...	...	...
1½	....	2½	22	...	...	2¾	3⅛	3½	...
1⅜	....	2¼	21	2¼	2½	2⅝	3	...	3⅞
1¼	....	1¾	20	...	2⅜	...	...	3⅜	3¾
1⅛	....	1¼	19	2	...	...	2⅞	3¼	3⅝
1	....	1	18	...	2¼	2½	...	...	...
⅞	...	¾	17	1⅞	2⅛	2⅜	2¾	3⅛	3½
...	..	....	16	...	2	2¼	2⅝	3	3⅜
			15	1¾	1⅞	...	...	2⅞	3¼
			13	...	...	2	2½	2¾	3
			12	1⅝	1¾	1⅞	2⅜	2⅝	2⅞
			11	1½	1⅝	...	2¼	2½	2¾
			10	...	...	1¾	2	2⅜	...
			9½	1⅜	1½	1⅝	...	...	2⅝
			9	...	...	...	1⅞	2¼	2½
			8½	...	1⅜	1½	...	2⅛	...
			8	...	...	...	1¾	2	2⅜
			7	1¼	...	1⅜	1⅝	1⅞	2¼
			6	...	...	...	1½	1¾	2
			5½	1⅛	1¼	...	1⅜	1⅝	1⅞
			5	...	1⅛	1¼	...	1½	1¾
			4½	...	...	...	...	...	1⅝
			4	...	...	1⅛	1¼	1⅜	1½
			3½	...	...	...	...	...	1⅜
			3	...	...	...	1⅛	1¼	...
			2	...	...	...	...	1⅛	1¼
			1½	...	...	...	...	...	1⅛

Sheaves and barrels should be about 30 times the circumference of the ropes used.

NOTE.—For shaft-winding at a high speed, one tenth of the breaking-strength of a rope is sometimes taken as fair working-load. For inclines, the proportion of load to breaking-strength varies according to gradient conditions, and friction should be allowed for.

began using it in St. Louis. This test also indicates the uniformity of the wire. If, after rupture, it be hooked up again and then runs a considerable time before breaking, it argues a weak spot in the wire which caused the first break. If, on the contrary, the first rupture is quickly followed by others on further continuance of the test, it indicates that the wire is uniformly worn out or fatigued, and that it was very uniform in quality.

From examinations the author has made on worn-out street-railway cables, he has reached the conclusion that the surfaces of contact with the car-grips become highly heated immediately on the rubbing-surface, and the resulting local expansion and contraction soon wears out or fatigues the metal just under these wearing-surfaces, thus causing the wires to become very brittle when bent with these surfaces on the extended side. Certainly this extreme brittleness exists at these points, causing the outer wires to become all broken up into short pieces, as shown in Fig. 621, before the rope finally fails. A shop-test could probably be devised which would determine approximately the relative resistance of wires to this kind of action. Running ropes of pulleys of too small a radius, thus stressing the outer wires to or beyond their elastic limits at every passage, would probably produce similar results.

453. The Albert-lay Rope (commonly called Lang-lay*).—By laying up the strands in the same direction as the wires are laid in the strand, the rope presents the appearance shown in Fig. 618. Any given outer wire remains now on the surface through a much greater distance, and the wires wear so as to make a rope almost as smooth as a solid rod, as shown in Fig. 619. Such a rope is best suited to running on or near the ground, or where there is a large amount of grinding surface-wear, as is the case with tail-ropes in mines, with inclines, and with tramways. Such a lay makes a more flexible rope also, and larger wires may be used for running over a given size of pulley.

* First used by Prof. Albert, of Clausthal, in 1834. It was patented, however, by Lang in 1879 and now commonly bears his name. J. Bucknall Smith in *Mining Journal* articles, June and July, 1896.

CHAPTER XXXIV.

THE MAGNETIC TESTING OF IRON AND STEEL.

By W. A. LAYMAN, M.S.

MAGNETIC PROPERTIES DEFINED.

454. Introductory.—Hardly less in industrial importance than the accurate determination of the mechanical properties of iron and steel is the careful testing of their magnetic properties. This arises from the double consideration of the vital part played by these properties and the immense consumption of iron and steel in what may broadly be termed the electrical manufactures. In the construction of electric dynamos, motors, transformers, and other forms of electrical machinery, there have gradually been evolved as clearly defined and as rigidly limiting requirements for iron and steel along the lines of magnetic *permeability* and magnetic *reluctance* or *hysteresis* as are specified by the mechanical engineer in the directions of elastic limit, ultimate strength, etc. These requirements are the outcome of constant endeavor to lessen the cost of manufacture and increase the operating efficiency of electrical apparatus. The use for dynamo-magnets of iron or steel possessing high *permeability* accomplishes the several good ends of lessening size, weight, and magnetizing energy required. The use of iron in transformer construction with low *hysteresis* losses means the same economy in this form of apparatus. Accordingly, iron manufacturers as well as electrical engineers are giving much attention not only to the testing of iron and steel that the magnetic properties of any given material may be known, but also to the study of the physical and chemical conditions which have a bearing on these properties, in order that a scientific manufacture of iron and steel for electrical work may be developed. Furthermore, the necessity for testing arises from another direction. The design of electrical machinery presupposes definite magnetic properties, in every given case, of the iron and steel employed. But it does not follow that the materials received for use can be depended upon to possess these qualities. On the contrary, they may vary between wide limits in a given quantity which is commercially of uniform quality. A single casting of steel or iron may vary greatly in different parts. The same is true of wrought iron, whether rolled in bars or sheets.

Consequently it is vitally necessary to the successful fulfilment of a designer's predictions that his iron and steel be thoroughly and intelligently tested. Otherwise his machine may fall far short not alone in operating efficiency, but in ability to carry its rated load.

455. The Magnetic Properties to be determined in testing, in the case of any given specimen of iron or steel, are the *permeability* and the *hysteresis*. Permeability is expressed as a ratio, and is the magnetization per unit of area, produced, divided by the magnetizing force producing it. For magnetizing force the conventional symbol of **H** is used, and for magnetization the symbol **B**. Both are magnitudes with reference to unit area. In the C.G.S. system this unit is the square centimeter, and in the English system the square inch. Accordingly, permeability in magnitude is $\frac{\mathbf{B}}{\mathbf{H}}$, and this magnitude is symbolized by the Greek letter μ.

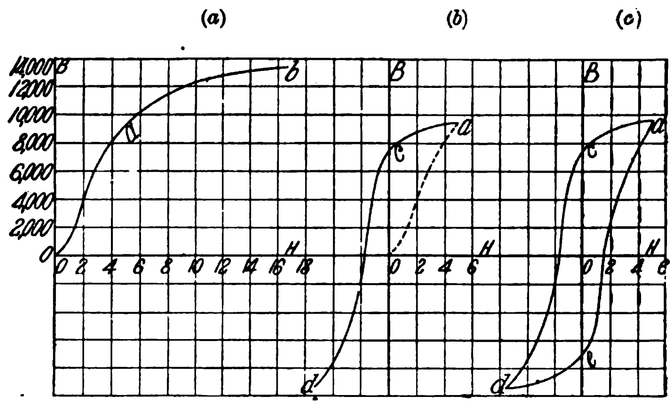
Imagine a soft iron bar closely wound from end to end with a magnetizing coil of insulated wire, the turns of which are uniformly distributed along the bar. When an electric current is sent through the coil a condition of magnetization is set up in the bar. This condition is numerically expressed by the number of *magnetic lines of force* per unit area of a section of the bar near its centre of length, or as **B**. The magnetizing force is numerically expressed by the number of magnetic lines of force per unit of area of the enclosed column of air which would take the place of the bar if it were removed, or as **H**. In other words, **H** is the magnetization produced in air, or **B** for air is equal to **H**, and μ for air is unity. **H** is determined by calculation from the formula

$$\mathbf{H} = \frac{0.4\pi CN}{L},$$

where N is the number of turns in the magnetizing coil, C the current in amperes passing through the coil, and L the length of the coil in inches if **H** is expressed per square inch, or in centimeters if **H** is expressed per square centimeter. **B** is determined experimentally. Permeability is not a constant in magnitude, as will be seen from a typical magnetization curve shown in (*a*) of Fig. 622. For example, when **H** is 4, **B** is 8000, or μ is 2000; but, when **H** is 16, **B** is only 13,000, or μ 812.5. This means that the magnetization can be intensified beyond a certain point only at the cost of a rapidly multiplying magnetizing energy.

456. Hysteresis is that property of all forms of iron and steel manifesting itself as a reluctance of the magnetization to follow changes in the magnetizing force. The iron bar above may be used to illustrate. If the bar, in its original state, possessed no appreciable magnetization, a constantly increasing magnetizing force would produce a magnetization following the dotted curve in Fig. 623 (*b*). If at the point *a* the magnetizing force begin to decrease, pass through zero, and increase in the negative direction to the

value of a positive, the curve instead of returning on itself would follow a new path acd in (b) of Fig. 622. If a cyclic operation be performed, the curve of magnetization would become a closed loop as in Fig. 622, (c). A study of this curve as compared with the same specimen's magnetization curve would reveal a constant dragging of the magnetization behind the magnetizing force. This dragging when magnetization is periodic involves an expenditure of energy, the enclosed area of the **B–H** loop as in (c) of Fig. 622 multiplied by $\frac{1}{4\pi}$ numerically expressing this work in ergs per cubic centimeter of the metal per cycle. The true cyclic state is not set up at once, but requires several repetitions of the cyclic operation, it having been found that the original magnetic intensity of a, Fig. 622, (c), is not at first entirely re-



CURVE OF MAGNETIZATION. CURVES OF HYSTERESIS.

FIG. 622. — Curves illustrating Magnetic Qualities of Iron. (*Inst. Civ. Engrs.*, vol. CXXVI.)

stored at the end of the cycle. It is evident now that the area of this hysteresis loop depends on the intensity of magnetization produced. In other words, for every maximum such as a there will be a definite **B–H** loop. It has been experimentally proved that the locus of a throughout the range of magnetization is the magnetization curve. This last fact will explain the practice hereafter brought out of obtaining the magnetization curve by subjecting the iron to reversals of magnetizing force and taking half the change of magnetization as equivalent to the magnetization which the same force would produce in a previously unmagnetized piece. Hysteresis losses waste themselves in the production of heat within the material.

All kinds of iron and steel exhibit this property in greater or less degree. The softer the specimen of any given material the less in general its hystere-

sis. For transformer and armature work, accordingly, it is of prime importance to carefully anneal the plates used. It may here be asked why, in work of this class, thin plates are used. The reason is that a cyclic current not only sets up a cyclic magnetization in the iron, but also an induced or "eddy" current in a path parallel to that in which the magnetizing current flows. The iron is therefore split up in thin plates in a transverse direction to this flow, and the surface oxide of these plates, together with the air-gap thus introduced, depended on to so greatly increase the resistance to flow of these "eddy" currents that they become of small importance, practically confining themselves in length of path to the thickness of the plate alone. Plates for this class of work are rolled in thicknesses varying between .014 and .035 inch.

The effort has been made to establish a general law by which, the hysteresis losses in any given material at any given magnetic *induction* or magnetization being known, the losses at any other induction could be calculated rather than determined experimentally. Exhaustive experimental results by Mr. C. P. Steinmetz established the conclusion on his part that, within the limits of magnetization employed in general practical work, the energy loss by hysteresis increases very closely with the 1.6 power of the magnetization. By means of this fact, commonly known as Steinmetz's Law, and the further fact that the hysteresis losses per unit of time increase in direct proportion with the increase of rapidity of the cyclic operation, a reasonably accurate calculation of the hysteresis losses under any set of conditions may be made, providing there is an experimental starting-point from which to work.

METHODS OF TESTING.

457. Measurement of Permeability.—There are in general four classes of experimental methods of measuring permeability:

(1) Magnetometric.
(2) Balance.
(3) Inductive.
(4) Traction.

Of these (1) and (2) are essentially laboratory methods. In (1) the specimen is made up as a bar, surrounded by a magnetizing coil. **B** is determined from observations of the deflections produced by the bar in a magnetometer. In (2) the operation is in general the same with the exception that a balancing magnet is used to neutralize the effect of the specimen under test on the magnetometer. The inductive and traction methods are, however, of such character as to permit of more general application. These may be considered somewhat in detail.

458. Inductive Methods.—These are based upon the general principle that an electric current will be induced in a given closed path if that path or any part of it is made to sweep across a magnetic field. In addition to

the magnetizing coil, there is wound upon the specimen of iron or steel a small "exploring" coil. The object sought in all the forms is the sudden removal of this exploring coil from the magnetic lines of force embraced by it, or, *vice versa*, the removal of the magnetic lines from the coil. This form of test is made upon a specimen ring or long bar of the iron or steel to be examined. The variations employed are all attempts to simplify the straightforward and accurate form in which the ring is used.

The Ring Method.—In this method the sample ring is wound with a *primary* or magnetizing coil and a *secondary* or exploring coil, each of a known number of turns. The general arrangement of apparatus for such a

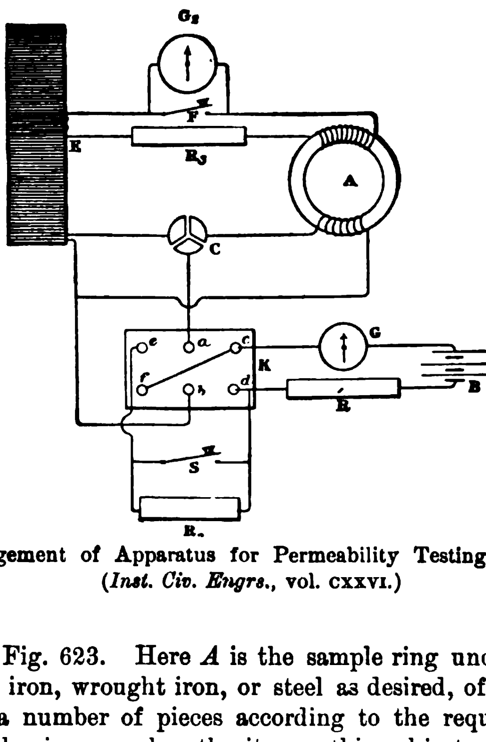


FIG. 623.—Arrangement of Apparatus for Permeability Testing by Ring Method. (*Inst. Civ. Engrs.*, vol. CXXVI.)

test is shown in Fig. 623. Here A is the sample ring under test. It may be either of cast iron, wrought iron, or steel as desired, of a single piece in thickness or of a number of pieces according to the requirements. Prof. J. A. Ewing, who is general authority on this subject, suggests that the width of the section of the ring measured radially be small as compared with the mean radius. He recommends an external diameter of 3 or 4 inches, with a radial thickness of about ¼ inch. B is a storage cell for supplying magnetizing current; C a two-way switch, in one position of which the current flowing will pass into the magnetizing coil of A, and in the other into the primary coil of E; E is an induction-coil, wound on a non-magnetic core, the secondary coil of which corresponds to the secondary or exploring coil of A and consists of but a few turns located near the centre of the

primary winding; F is a short-circuiting switch for the D'Arsonval * ballistic galvanometer, being employed merely to bring the needle of the galvanometer to rest after an observation has been made; G is a current-meter; K a reversing switch by means of which the direction of flow of current in the primary winding of A or E may be reversed at will, and also by means of which in one position the auxiliary resistance R_2 may be cut into circuit in case the short-circuiting switch S across it is open.

This arrangement can be used for determining either the magnetization curve or the hysteresis loop. The method of procedure in the former case is as follows: A definite current, as shown on G, is passed from the battery through the primary coil of A, S and F being closed. When a general condition of rest is established for both G and G_2, by means of K the current is several times reversed. On the final reversal F is opened and the swing of G_2 noted. Half of this swing is taken as representing the magnetization produced by the current read. The object sought in the above operation is the complete removal of the magnetic lines of force from the specimen. Were the current not reversed in direction such would not be the case owing to hysteresis, and the galvanometer would only indicate the removal of the difference in magnetic lines between that produced by the given current and that of the residual charge remaining after the magnetizing force has been entirely removed. By reversing the current instead of cutting it off the whole magnetic charge is removed and at once reinserted, or the effect on the exploring coil has been equivalent to that of the removal of twice the maximum number of lines of force.

The value of **B** so determined is translated from the scale of the galvanometer by standardizing the instrument with E. A given current is sent through the primary of E and then suddenly cut off. The deflection on the galvanometer thereby resulting, due to a current being generated in the exploring coil of E, is noted. The magnetizing force **H** has by this operation been made to record itself on G_2. **H** is at once calculated by the formula above given of $\mathbf{H} = \frac{0.4\pi CN}{L}$, N being the number of primary turns of E, L the length of E, and C the current observed in amperes. From this value of **H** a constant for the galvanometer scale is determined. This constant †

* This form of galvanometer consists of a coil swinging between the poles of a strong horseshoe magnet. The swinging coil carries a mirror, and scale deflections are read with a telescope in the usual manner. In damping, the swinging coil is short-circuited by S, and the current then generated in the coil by the swing quickly brings it to rest.

† The deflection produced when any test is made whether on E or A is proportional to the product of the induction per unit of area multiplied by the area multiplied by the turns of the exploring coil multiplied by a constant. In the case of two tests, one on A and one on E,

$$\frac{d'}{\frac{d''}{2}} = \frac{\mathbf{H}a't' \text{ constant } C}{\mathbf{B}a''t'' \text{ constant } C}$$

K, which is the induction **B** in the specimen ring for a deflection of one division on the galvanometer scale when a test on A is made, is $K = \frac{\mathbf{H}a't'}{2a''t''d'}$, where H is the induction per unit of section of the core of E, a' the area of the core of E, t' the number of turns in the exploring coil of E, a'' the sectional area of the specimen ring A, t'' the number of turns in the exploring coil of A, and d' the divisions deflection on G_2 when the primary circuit of E was broken.

The calibration of G_2 determined, the induction **B** in a given test on A would be the constant K multiplied by the divisions deflection shown on the galvanometer scale. **H** for the magnetizing coil of A with any current is calculated from the formula for **H** as given. The plotting of the entire magnetization curve then but requires a series of tests on A alone, the current being varied.

The Divided-bar Method.—This method, due to Dr. Hopkinson,* is an attempt to secure the same results as in the ring method with a much cheaper and more easily made specimen. (See Fig. 624.) Here a heavy block of annealed wrought iron Y has its central portion cut out to receive a magnetizing coil C. The test sample of iron or steel SP is made in two parts, carefully turned. These parts slide closely through holes bored in the ends of the block, and the trued ends meet at the exploring coil E. B is a battery for supplying current, A an ampere-meter, S a reversing switch, R an adjustable resistance, and BG a ballistic galvanometer. In operation the specimen rods are pushed tightly together and magnetized to any point on the sought-for magnetization curve. Simultaneously the circuit is now broken and the rods pulled apart. The rods separated, a spring at the same instant pulls the exploring coil E entirely out of the magnetic circuit. The effect is that of the entire removal of the lines of force from the exploring coil, and the whole deflection of BG is accordingly a measure of the induction **B** when BG has been properly calibrated. Dr. Hopkinson's idea was to make the magnetic resistance of the soft iron block so small, as compared with the rods, that the condition of the iron circuit would be very nearly the same as if the outside ends of the rods were together. His object in making the specimen in two pieces was to afford a chance of getting the exploring coil out, otherwise the magnetic sluggishness of the yoke would have vitiated the re-

$$\mathbf{B} = \frac{\mathbf{H}a't'd''}{2a''t''d'}, \text{ or}$$

$\frac{\mathbf{B}}{d''} = \mathbf{B}$ per scale division of deflection in test on $A =$

$$K = \frac{\mathbf{H}a't'}{2a''t''d'}.$$

In this discussion d' and d'' are deflections respectively on E and A, and the factors $\mathbf{H}a't'$ refer to the test on E, while $a''t''\mathbf{B}$ refer to A, as above. The factor 2 enters in by reason of the deflection d'' being made as above explained.

* *Phil. Trans.* p. 456, 1885.

sults of the tests. The disadvantage involved is the introduction of a very thin air-gap where the rods meet and also where they pass through the yoke. This introduces an error in the calculation of the true **H**, for it is evident that the magnetizing current must consist of four factors: (1) that required to overcome the resistance of the yoke with the given induction **B** in the

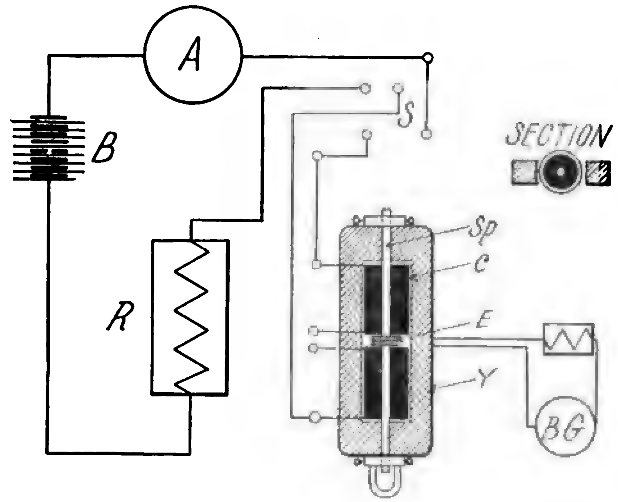


Fig. 624.—Hopkinson's Divided-bar Method. (Thompson.)

rod; (2) that required to overcome the resistance of the air-gap between rods and yoke; (3) that required to overcome the break in the rods; and (4) that required to overcome the resistance of the rods themselves. (1) may be negligible, but (2) and (3) are not.

The Double-bar Method.—To overcome the objection to Dr. Hopkinson's arrangement, Prof. Ewing has devised a double-bar double-yoke method in which the errors involved in the divided-bar method can be experimentally determined and allowance then made for them. This is roughly illustrated in Fig. 625. The specimen to be tested now consists of two bars, magnetized in opposite directions and by equal magnetizing forces, and united by short two-part yokes at each end. The two-part yokes, by means of the clamp-screws, tightly clamp the specimen bars. The test is made in two parts. First the full length of the bars is used, as shown in part (*a*) of Fig. 625. The value of **B** is determined ballistically, and the magnetizing force H', error included, calculated. The second test is made with the clear length of the bars between the yokes reduced to one half, and shorter coils used as shown in part (*b*) of the figure. Now the value of the magnetizing force H'' is determined for the same value of **B** as found in the first part of the test. The error in the second test is just twice that involved in the first, and the correction it is necessary to subtract from H' to give the true magnetizing force for the value of **B** determined in the first trial is $H'' - H'$.*

* In the first trial $HL_1 = 0.4\pi C_1N_1 = \mathbf{H}L_1 + E$, where **H** is the true magnetizing

The Magnetic Bridge Method.—With the object in view of still further facilitating and simplifying permeability testing to meet workshop requirements, Prof. Ewing has devised another very neat method which he calls the "magnetic bridge." The principle involved in this method is the production

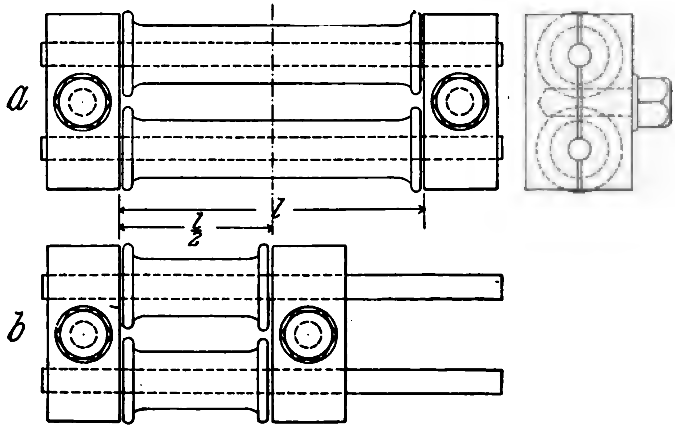


Fig. 625.—Ewing's Double-bar Permeability Method. (*Inst. Civ. Engrs.*, vol. cxxvi.)

of a magnetic balance, so to speak, between a test specimen and a ***standard*** specimen, the exact permeability curve of the latter having been previously determined by the two-yoke ballistic method above. The process is analogous to resistance measurements with the Wheatstone bridge, hence Prof. Ewing's suggested name of the magnetic bridge. The arrangement is illustrated in Fig. 626. The two bars, one the test specimen and the other the standard, are *a*, *a*. Connecting these bars at their ends are heavy yokes of soft iron *bb'*, made in the form of rings and held in place by three longitudinal brass rods *fff*. A cross yoke of soft iron *gg*, with a central break in it at *h*, is carried up above the end yokes. In the gap *h* a detector-needle is inserted, this being directed by an adjustable magnet *k* on a brass rod below it. The

force, C_1 the current read, N_1 the turns, L_1 the clear length of the specimen bars, and E the error introduced by the joint with the yokes and the yoke resistance.

In the second trial $H'L_2 = 0.4\pi C_2N_2 = \mathbf{H}L_2 + E$, where L_2, C_2, and N_2 have corresponding values. Since $\mathbf{B}$ is the same in both trials, $\mathbf{H}$ is also the same. For this reason E is the same in both cases.

From the above

$$H' = \mathbf{H} + \frac{E}{L_1} = \frac{0.4\pi C_1N_1}{L_1},$$

and

$$H'' = \mathbf{H} + \frac{E}{L_2} = \frac{0.4\pi C_2N_2}{L_2},$$

from which $H'' - H' = \frac{E}{L_2} - \frac{E}{L_1} = \frac{2E}{L_1} - \frac{E}{L_1} = \frac{E}{L_1}.$

Accordingly, $\mathbf{H} = H' - \frac{E}{L_1} = H' - (H'' - H').$

bars *aa* are enclosed by magnetizing coils wound on brass spools. These coils are so arranged with switching devices that fractional parts of them may be cut in or out of circuit at will. The test consists in sending a given current into these coils connected in series. If the magnetizations produced in the bars are equal, there will be no difference in *magnetic potential* between the yokes *bb'*. But if the magnetizations are unequal, there will be

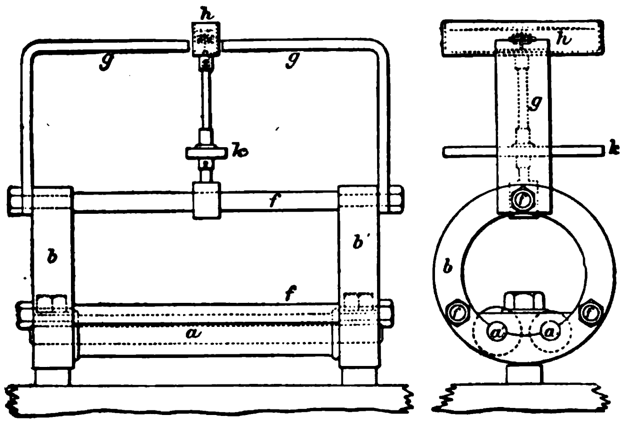


FIG. 626.—Ewing's Magnetic Bridge. (*Inst. Civ. Engrs.*, vol. CXXVI.)

a difference in magnetic potential between *b* and *b'*, and this will manifest itself by endeavoring to relieve itself across the soft bar *g*, thereby producing a deflection of the detector-needle. The magnetizing turns over the two bars *aa* are now so adjusted with reference to each other that, on reversal of the current, no permanent displacement of the detector-needle is observed. The ratio of the two magnetizing forces is then the ratio of the number of effective turns employed. **B** for the standard bar is taken from a table accompanying it, the ampere-turns of magnetization being known. The permeability for either bar is easily calculated. It is here assumed that the measured magnetizing forces are in the same ratio as the real forces, an assumption which involves no appreciable error.

The idea of comparing permeabilities in the two arms of a magnetic circuit is not new,* but Prof. Ewing is the first to employ this adaptation. The only trouble that seems to have been experienced in using this device is a "kick" on the part of the detector-needle arising from different periods of time required for the magnetizations in the standard and specimen bars to establish themselves.

The Voltmeter Method.—This method, as illustrated in Fig. 627, has been used by Prof. W. E. Ayrton. The specimen to be tested is made up as a bar and slipped into the heavy pole-pieces, a magnetizing coil wound on a bobbin sliding over it between the poles. A small armature revolves

* *Trans. Amer. Inst. Elec. Eng.*, vol. LX. p. 3.

between the poles at a constant speed, this armature being driven by a small motor. When the specimen is magnetized, the armature generates an electric current which flows through the voltmeter. This current is directly proportional to the induction in the bar. From the ammeter-reading the magnetizing force **H** is known, and from the voltmeter-reading the induc-

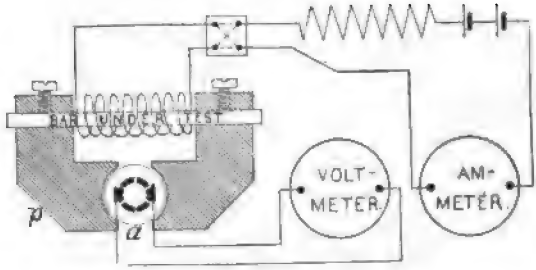


FIG. 627.—Ayrton's Voltmeter Method. (*Inst. Civ. Engrs.*, vol. CXXVI.)

tion **B** is determined by comparisons made with a standard bar employed under exactly the same conditions as to magnetizing force and speed. The magnetic curve of this standard bar has been previously determined by a ballistic method.

459. Traction Methods.—These methods are based on the fact that when magnetic induction crosses, through surface-faces in close contact, from one magnetized body to another, these bodies resist being pulled apart. The amount of this resistance or "tractive force" is dependent on the intensity of the induction. The traction methods are all directed to the simplifying of induction measurements. Of them all, perhaps the best known and most generally used is Prof. S. P. Thompson's "Permeameter" method. This is illustrated in Fig. 628. In general the apparatus closely resembles Dr. Hopkinson's divided-bar arrangement. There is the same heavy yoke and a single magnetizing coil. The change is largely in the test-specimen, which is now made as a single rod, carefully faced on the lower end where it makes close contact against the yoke. When a current is sent through the magnetizing coil, the rod sticks tightly to the yoke at its lower end. This tractive force is measured upon the spring-balance in pounds pull required to separate the rod from the yoke. **B** is deduced from the formula $\mathbf{B} = 1317\sqrt{P \div A} + \mathbf{H}$, where A is the area of contact of the rod upon the yoke, and P the pull in pounds. **H** is here added for the reason that the magnetizing coil is not moved with the rod, as a consequence of which the pull is that due to $\mathbf{B} - \mathbf{H}$ lines.

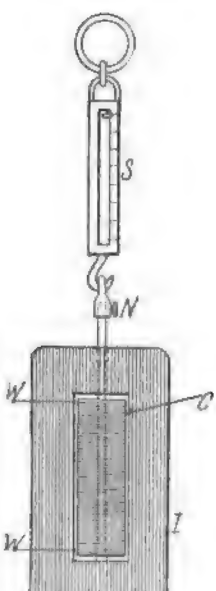


FIG. 628. — Thompson's Permeameter. (Thompson.)

The Magnetic Balance.—This device, due to Dr. H. du Bois, is illus-

trated in Fig. 629. The yoke, as will be seen, is divided, the upper portion being supported on knife-edges in such a way that an air-gap of definite width is introduced between portions of the yoke. The test-bar is inserted between the lower detached ends of the yoke. The test is made by moving the sliding weights until the rocking part of the yoke is pulled away from

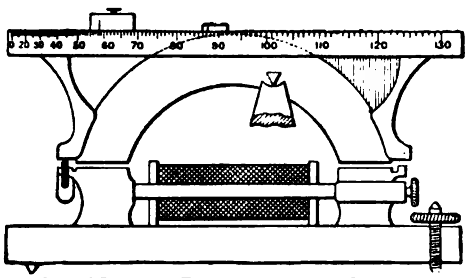


FIG. 629.—*Du Bois Magnetic Balance.* (*Inst. Civ. Engrs.*, vol. CXXVI.)

the small stop which determines the air-gap. A curve is furnished by the maker of the apparatus for correcting **H** to compensate for the errors introduced by the air-gap.

460. Measurement of Hysteresis.—In the measurement of hysteresis losses a great variety of methods have been used. In general these have aimed at simplicity and facility of testing rather than accuracy. As a result they have been more nearly relative than absolute methods. Some have been based on the fact that the hysteresis losses waste themselves in the production of heat within the iron. In such devices there is always liable to be a great inaccuracy arising from the complication of heat being also produced by the eddy currents which a cyclic magnetizing current will set up If the test-specimen is made of very thin plates, and if the reversals of magnetizing currents are not too rapid, this error may be largely eliminated.

Another series of methods involves the use of an electric wattmeter. Here the specimen usually takes the form of a closed magnetic circuit, wound with a strong magnetizing coil into which an alternating current is sent. By means of the wattmeter the total energy consumed in producing the magnetization is observed. Here, again, however, the complication of eddy currents enters, as also a loss of energy due to the resistance of the wire. This method is nevertheless a very common one in workshops where a systematic and strictly scientific study of materials has not been entered into. By means of an exploring coil to which a voltmeter is attached the induction **B** is known. In the hands of a man who understands thoroughly the use of electrical instruments, and who also has a knowledge of the relations existing between power, pressure, and current in a circuit carrying an alternating current, a wattmeter method will afford no small degree of satisfactory service. But in the hands of a less able man it is of little value.

The Ring Method.—All other methods failing, there remains the straightforward ballistic or ring method shown in Fig. 623 and by which perme-

ability testing was illustrated. For the plotting of the accurate hysteresis curve the general method of procedure is but slightly changed. The first step is to determine accurately the maximum point a of the loop, Fig. 622 (c). This is done exactly as the corresponding point on the magnetization curve in Fig. 622 (a) would be found. After a has been determined, the switch K being on the proper side to permit it, the magnetizing current for a is suddenly reduced by any desired amount by simply opening the switch S and thereby cutting in R_2. The magnetization drops in magnitude as does the current, but not so far. The full swing of the galvanometer measures its change. This would determine a single point between a and c, Fig. 622 (c), dependent on R_2 for its position. Then K is switched over to the other side, giving again the full current of a, but reversed in sign and now the negative current of d. Next R_2 is changed in value, and then, with S still open, K is switched again. The current becomes positive in direction, but decreased from the magnitude of d by an amount depending on R_2. The swing of the galvanometer this time determines a point somewhere between a and d. Closing S, the magnetization runs back again to a, and everything is in readiness for another cycle. Thus, with each cycle, two points on the loop are found, one between a and c, and the other between e and a. As the curve is symmetrical, ac is also de, and ea also dc. Thus the full curve is established.

Ewing's Hysteresis-tester.—Prof. J. A. Ewing has recently brought forward an extremely simple machine for direct measurement of hysteresis which he calls a "hysteresis-tester." This is illustrated in Fig. 630. The sample of iron which is to be tested by comparison with a standard sample is prepared by piling about half a dozen $3 \times \frac{5}{8}$-inch stampings or strips of the iron sheet into a bundle, and clamping the same between vulcanite washers by the clamps bb on the carrier a.

This carrier is made to revolve by means of d between the poles of a strong permanent magnet e, which magnet is hung on knife-edges in line with the axis of the carrier so that it may swing in a concentric arc with the carrier a. The magnet is given some stability by a small weight g. Below e is a small cup in which a suspended vane swings in oil, thus providing a dash-pot.

The principle involved is that the hysteresis gives rise to a mechanical couple between the sample and the magnet, this couple tending to pull the magnet around in a circle after the revolving specimen. A deflection from a vertical plane results, which deflection is indicated on a scale at the top of the supporting post of e. This deflection is a measure of the hysteresis. With reference to the operation of the instrument Prof. Ewing says: "The deflection is independent of the speed (so long as that is not so high as to cause supplementary deflection by air-currents), and hence no particular care has to be taken to turn the handle at a uniform rate. The operator has merely to turn the handle just fast enough to make the impulses which are given at each half-revolution blend into a steady deflection. The deflec-

tion is observed first to one side and then to the other by reversing the direction of rotation." A considerable air-space is left between the ends of the sample and the magnet-poles for the purpose of putting practically all of the resistance of the magnetic circuit in these gaps, and consequently eliminating the permeability of the specimen as a factor. The induction used in testing is about four thousand lines per square centimeter. Prof.

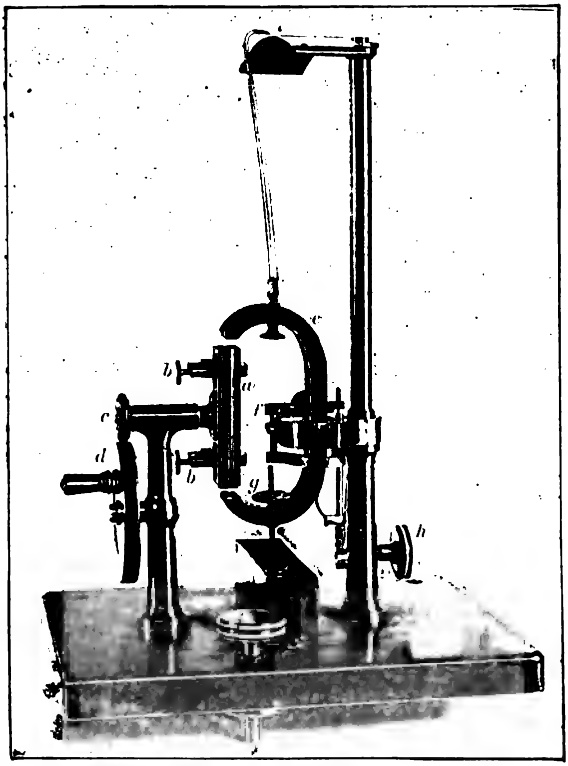


FIG. 630.—Ewing's Hysteresis Tester. (*Inst. Civ. Engrs.*, vol. CXXVI.)

Ewing has found that no exact adjustment of the section of the sample is necessary, it being sufficient to take that number of strips which come nearest in weight to the standard sample which is furnished with the instrument. "A small error is involved, probably due to the fact that there is some hysteresis in the magnet itself when the sample is revolving."

The objection to the instrument is that tests can only be made at a single induction. To this objection Prof. Ewing has replied that Steinmetz's Law, within the range of inductions usually obtaining, affords easy translation to any induction desired. All in all the instrument is certainly a valuable addition to the magnetic testing apparatus now at command.

RESULTS OF TESTS

461. Development Due to Testing.—Magnetic tests have established relationships between the various forms of iron and steel manufactured, which are depicted in the typical curves of magnetization and hysteresis presented in Figs. 631 to 635 inclusive. These curves are worthy of close study, as from them may be read not only the characteristic magnetic differences between the various forms of steel and iron, but also the story of many radical changes that have quickly succeeded each other in the manufacture of electrical machinery. Fig. 631 alone, for example, will explain why the

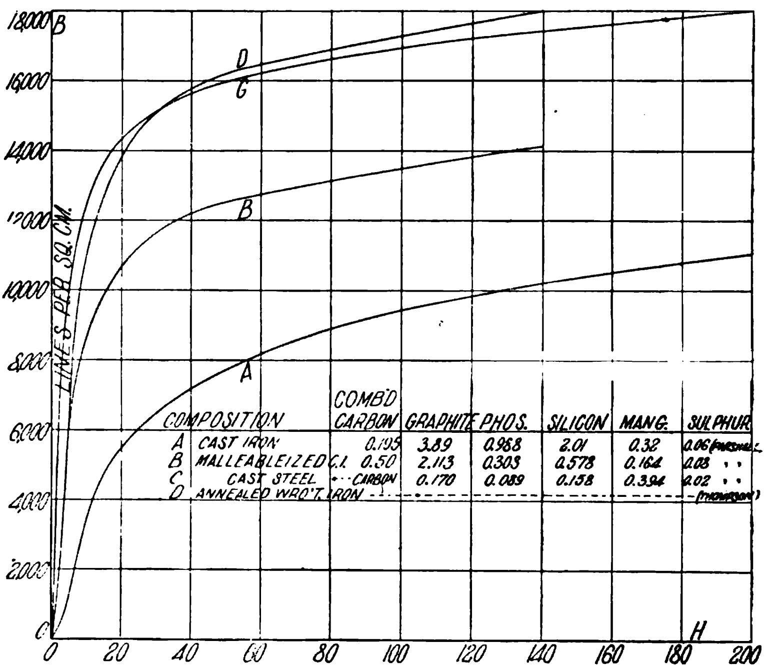


FIG. 631.—Characteristic Magnetization Curves.

generating dynamo of to-day, with only from one half to three quarters the volume of magnetic material in it, is equal in electrical capacity to the dynamos of ten years ago. This fact is evident from the magnetic superiority of cast steel over cast iron. In the early manufacture of dynamos and motors, the field-magnets and connecting yokes were made entirely of cast iron. The magnetic superiority of wrought iron was soon more generally appreciated, and pole-pieces were then forged, where manufacturing facilities permitted. This was a distinct gain, but as a considerable part of the magnetic circuit was still of cast iron because all parts of the dynamo frame, for mechanical reasons, could not be forged, much was still to be desired from the standpoint of magnetic economy. Forged steel, having no

magnetic advantage over wrought iron, afforded no improvement. Cast steel, however, developing practically as high permeability as either wrought iron or forged steel, except with very weak magnetizing forces, at once accomplished the great advantage sought. The entire framework, pole-pieces and yokes included, could be easily cast, and the result has been the adoption, where facilities permit, of cast steel for all forms of electrical machinery calling for solid rather than laminated parts. Within the last two years a still further improved form of dynamo-magnet construction has been introduced, mention of which only can be made here. This is the building up of the magnetic polar projections of the field-frame of laminated wrought iron or steel. These poles are then placed in their proper relative positions in the mould for the field-frame, and the cast steel for the remaining parts poured around them. The object sought is the elimination of the eddy currents, which are induced in the pole-faces when the machine is in operation.

Fig. 632 will especially make clear why there is such a marked superiority

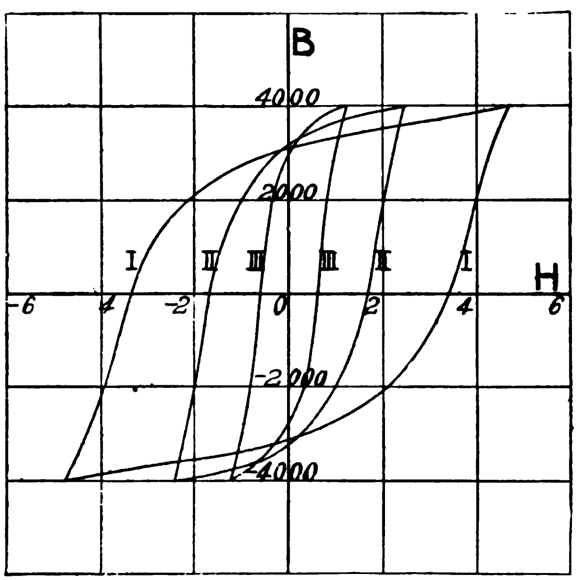


Fig. 632.—Curves showing Relative Hysteresis Quality of Three Specimens. (*Inst. Civ. Engrs.* vol. CXXVI.)

of one alternating-current transformer over another of the same general dimensions, a fact generally known but not generally understood. In this figure are given the hysteresis curves of three different makes of iron furnished for the same class of work. The tests were made by Prof. Ewing. XIV was furnished as soft wrought iron, but the curve discredits this claim. It will be seen that XII is vastly superior to XIII, while XIV as compared with either XII or XIII is exceedingly poor. Fig. 635 still further em-

phasizes how marked may be the differences between materials, under a cyclic magnetization, all supposed to be commercially suitable for the same class of work. Here are given both magnetization and hysteresis curves, the results being taken from a recent paper by Prof. Ewing.* I is a special grade of Swedish transformer iron, from which Prof. Ewing makes the standard bars used with his hysteresis-tester; II is a transformer-plate of steel; III, another quality of Swedish transformer iron; IV, a transformer-plate made from scrap-iron; and V, a specimen of iron wire used by Mr. Swinburne some years ago in the manufacture of his "hedgehog" transformer. From these curves is further evident the cause of much of the great improvement that has been made in the efficiency of all forms of apparatus where hysteresis is involved.

462. Conditions Affecting Magnetic Quality.—Careful testing has furnished much information as to the conditions which affect the magnetic quality of iron and steel. Of these conditions, which may in general be classed as physical, great magnetic differences in materials are sometimes due. For example, permeability is seriously affected by such operations as hammering, rolling, etc. A given specimen of soft annealed iron may have its permeability greatly reduced by the hardening resulting from stretching. In the same way, steel, hard drawn, has much lower permeability than steel annealed. With cast metal the suddenness of cooling, in similar manner, seriously affects the magnetic quality. Material under excessive strain also has its permeability lowered. As a general rule, the warmer the metal the lower its permeability. That the hysteresis-factor is also seriously affected by many of these same conditions is more generally appreciated. Hence the electrical manufacturer's effort to carefully anneal his iron.

The part played by chemical composition is not so well understood. The fact of the matter is that as yet the study of magnetic quality from the standpoint of chemical composition has not been systematically or exhaustively entered into. But few results of value in this direction are at command. Accordingly, a recent paper by Mr. H. F. Parshall † is of unusual value, as it contains results of a great many tests in which chemical composition was very closely considered. Mr. Parshall calls attention to the fact, as brought out in his results, that, "beginning with the most impure cast iron and passing through the several grades of cast iron, steel, and wrought iron, the magnetic properties accord principally with the amounts of carbon present, and in a lesser degree with the properties of sulphur, phosphorus, manganese, and other less usual ingredients; and that an excess of any one or of the sum of all the ingredients (other than iron) has a noticeable effect on the magnetic properties." Fig. 634 is here produced from Mr. Parshall's results to support his conclusions, as also to indicate the general magnetic qualities of the materials tested. In this direction it may be stated as a

* *Proc. Inst. Civil Engrs.*, vol. CXXVI. p. 184.

† *Proc. Inst. Civil Eng.*, vol. CXXVI. p. 220.

generally observed fact that the purer the iron the higher the permeability. Results of tests by Prof. Ewing in curves II, III, and IV of Fig. 633,

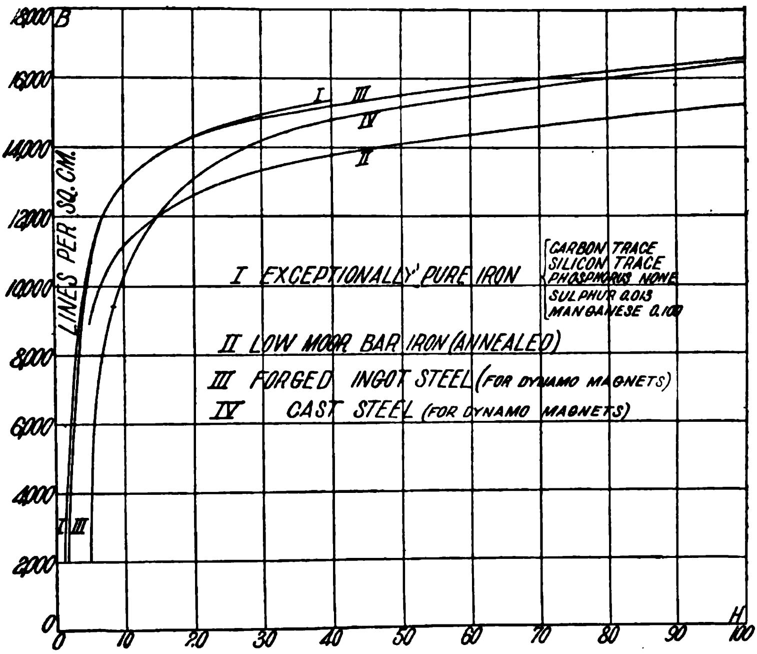


FIG. 633.—Magnetization Curves showing Relation between Wrought Iron and Steel.

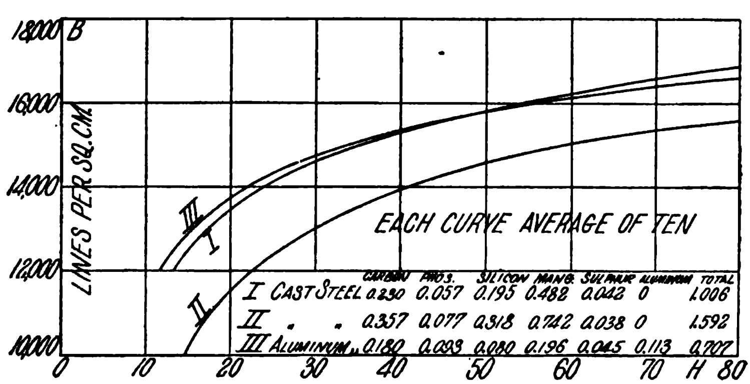


FIG. 634.—Parshall's Results showing Effect of Impurities on Magnetization.

although the chemical compositions are not given, may be said to bear out this fact.

With reference to hysteresis loss in laminated iron, there has recently been shown to exist in many qualities of material a gradual deterioration under the conditions imposed by continued service. Of such a striking

nature is this change that much attention is being paid to it. Cases are on record in which the hysteresis losses in transformers under heavy service have increased over 100 per cent. Mr. Parshall cites one case in which, during two years' service, the increase was about 200 per cent. Many observations bearing on this property of iron and steel are given by Mr. Parshall. His tests covered six months' service, in general, of the samples of material under examination. The magnetic induction was closely the same in all specimens, while the temperature of service varied. Several specimens gave no evidence of change whatsoever. Others changed as much as 25 per cent. Chemical analysis only indicated a difference in the percentage of silicon present, but whether the change is related to or dependent on the silicon cannot now be positively stated. Taking this change of quality into account, it may be seen that the manufacturer of transformers has two con-

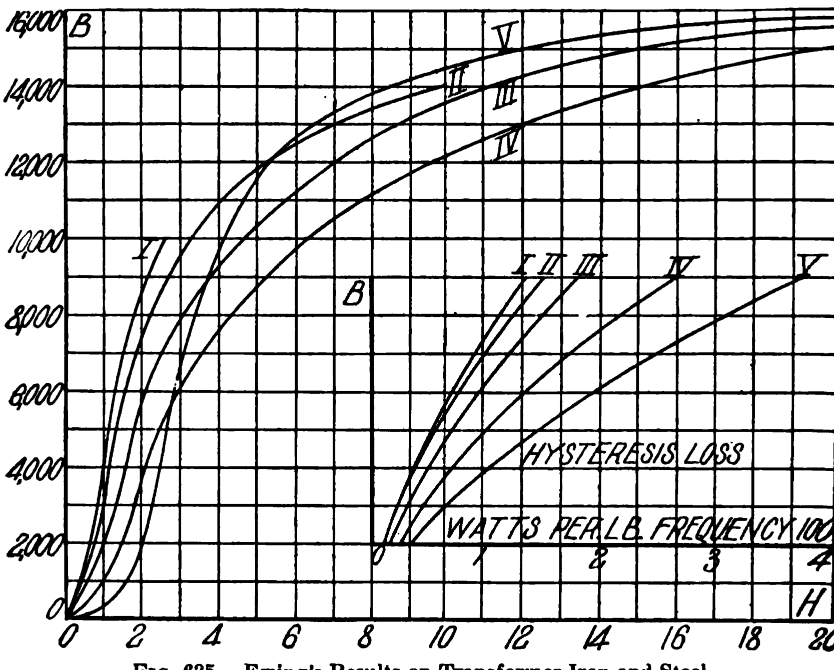


FIG. 635.—Ewing's Results on Transformer Iron and Steel.

siderations involved in the selection of his iron or steel plate. First, the quality of material with reference to permeability and hysteresis, and second the quality of material with reference to permanency of magnetic properties under continued service. It is possible for the change in quality to take from materials greatly superior, on first examination, all of the superiority possessed by them. More data are badly needed upon this phase of the subject.

463. In Conclusion the reader is again invited to closely study all the

curves presented. They are plotted from accurate and authentic tests, and show not alone the relative qualities and absolute magnetic values of the various forms of steel and iron, but also many elements which the special needs of the investigator may lead him to search for. Furthermore, the data are almost entirely new, which gives a still greater value.

It is also desired to place great emphasis on the need as well as desirability of thorough, accurate, and continued magnetic testing of materials.

TABLE LXI.—USEFUL DATA ON ELECTRICAL CONDUCTIVITY, ETC.

	Electrical Conductivity.			Specific Gravity.	Specific Heat. Average at Ordinary Temperature. (Regnault.)	Fusion Pts. in Degrees Fahr.	Coefficient of Thermal Conductivity. (Wiedemann and Franz.)
	At Normal Temperatures. (Lazare Weiler.)	Mathiessen. At 0° C. At 32° Fahr.	Mathiessen. At 100° C. At 212° Fahr.				
Pure silver	100	100	71.56	10.505	0.0570	1733 to 1873	100
Pure copper	100	......	......	8.853	0.0951	1929 to 1996	73.6
Refined and crystallized copper	99.9	99.95	70.27				
Telegraphic silicious bronze	98.0						
Alloy of copper and silver (50%)	86.65						
Pure gold	78.0	77.96	55.90	19.258	0.0324	1913 to 2285	53.2
Silicide of copper, 4% Si	75.0						
Silicide of copper, 12% Si	54.7						
Pure aluminum	54.2	......	......	2.67	0.2185	1157	37.96 to 38.87
Tin with 12% of sodium	46.9						
Telephonic silicious bronze	35.0						
Copper with 10% of lead	30.0						
Pure zinc	29.9	29.02	20.67	7.00	0.0956	680 to 779	
Telephonic phosphor-bronze	29.0						
Silicious brass, 25% zinc	26.49						
Brass, 35% zinc	21.5						
Phosphor-tin	17.7						
Alloy of gold and silver (50%)	16.12						
Swedish iron	16.0						
Pure Banca tin	15.45	12.36	8.67	7.35	0.0562	442 to 446	14.5
Antimonial copper	12.7						
Aluminum bronze (10%)	12.6						
Siemens steel	12.0						
Pure platinum	10.6	18.00	......	21.5	0.0324	3227	8.4
Copper with 10% of nickel	10.6						
Cadmium amalgam (15%)	10.2						
Drouier mercurial bronze	10.14						
Arsenical copper (10%)	9.1						
Pure lead	8.88	8.32	5.86	11.38	0.0314	608 to 618	8.5
Bronze with 20% of tin	8.4						
Pure nickel	7.89	......	......	8.8	0.10863		
Phosphor-bronze, 10% tin	6.50						
Phosphor-copper, 9% phosphor	4.90						
Antimony	3.88	4.62	3.26	2.67	0.0508	810 to 1150	19.2* to 21.5
Mercury	......	1.60	......	13.58	0.0333	....	67.7*

* Calvert & Johnson.

On the scientist's part this necessity is fully appreciated, for he understands that just as the present knowledge of their magnetic properties and the successful use of iron and steel in the electrical manufactures is the result of careful testing, so also is future development along these lines dependent on testing. On the manufacturer's part there is a constantly growing appreciation of the great advantages to be gained from testing. The electrical manufacturer, knowing how great may be the variation in magnetic quality of the materials furnished him, is keenly alive to the fact that he cannot maintain nor advance the standard of his apparatus without testing. The iron and steel manufacturer, in turn, is coming to see that he cannot much longer furnish for electrical work such materials as are included in his regular line of manufacture. Electrical work is beginning to demand rather than accept, and soon must have materials which are shown by scientific study to best meet the requirements of this department of industry. The situation is one in which the consumer must not only protect himself, but lend all the assistance within his power to the scientific study of magnetic phenomena, while the iron manufacturer must, by the limitations of trade competition, know from his own investigations the quality of the materials he is turning out and the part played by physical and chemical conditions of manufacture. From every point of view is clearly evident the importance of the magnetic testing of iron and steel.

APPENDIX A.

A BIOGRAPHICAL SKETCH OF THE LIFE OF PROFESSOR JOHANN BAUSCHINGER.*

PROFESSOR BAUSCHINGER was born in Nürnburg in 1834. His father was an artisan, and had a large family. Young Bauschinger, therefore, at an early age saw the earnest side of life, and he learned to have faith in his ability to support himself. At the age of fourteen he began giving private lessons. However, gifted with a strong will-power, he completed the course at the Polytechnic School with honors in 1853 at the same time receiving his certificate from the Latin School. Having selected mathematics and physics as the branches which he desired eventually to teach, he studied for three years at the University of Munich. Under von Lamont he studied with great enthusiasm theoretical and practical astronomy. He had the rare opportunity of using the astronomical and magnetic instruments in the royal observatory at Bogenhausen. Here he developed that faculty of keen observation and learned the scientific methods of discussing and reducing observations to obtain the most probable results, which afterward stood him in such good stead.

In the fall of 1856, after having passed the examinations for teacher in mathematics and physics, he accepted the position as assistant in physics and descriptive geometry at the Polytechnic School in Augsburg, and in 1857 he was called to Fürth, where he became teacher of mathematics and physics at the Royal Industrial School.

In 1866 Bauschinger was transferred to the Academy in Munich, and two years later he became professor there of mechanical engineering at the newly-founded Technical School. In 1870 he became director of the testing laboratory, which was built under his direction and according to his plans. Here he remained and labored in the interest of science until his death.

At Fürth Bauschinger had already developed a literary activity. Papers by him on subjects pertaining to mechanical engineering and thermodynamics appeared in various publications. As an independent work he published his popularly-written *School of Mechanics.* This was followed in 1871 by *The Elements of Graphical Statics.*

Without doubt Bauschinger's best and most valuable publications were those relative to his physical tests. His *Indicator Trials on Locomotives* had been begun as early as 1865, and they were continued at Munich under extreme difficulties on account of the great amount of work he was called on to perform as professor in the Technical School. However, Bauschinger's main field of investigation was in the testing of materials. Here he earned great and indisputable fame.

His writings since 1871, published in the *Journal of the Society of Bavarian Architects and Engineers* and other publications, and later in his *Communications from the Testing Laboratory of the School of Technology of Munich,* will long furnish ample data for the study of the strength of materials. His was the first public testing laboratory in all Germany, and it has continued to stand as the most noted in the world in many respects. This institution, with its furnishings, has served as a model for all later similar establishments.

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* Compiled by the author from a more extended memoir by Prof. A. Martens, and published in the *Digest of Physical Tests* for July, 1896.

Bauschinger greatly improved our means for testing materials by the invention of accurate measuring apparatus, one example of which was his application of the Gauss method of mirror readings, by which all measurements are very accurately obtained. Many other accurate measuring and testing devices, now commonly employed, are due to him.

Referring now to his contributions to the science of the strength of materials, in his "Communications" we may say, in short, that in the numbers 1, 4, 5, 7, 8, 10, 11, 18, and 19 he treats of the strength of cements, mortars, and artificial and natural building-stones. In numbers 1, 7, and 8 Bauschinger speaks of his numerous tests of cements, and cement and lime mortars. In these, as well as in the tests of artificial and natural stones discussed in numbers 4, 5, 10, and 11, 18, and 19, different methods were used, and the various results were compared. He has made a special study of the elasticity and strength of building-stones. At the same time and in the most detailed manner he made cross-bending, tension, compression, and shearing tests of the same materials.

For the investigation of the wearing of stones an abrasion method was devised (see Art. 436, p. 645). The effects of freezing are minutely examined and compared, and many simplifications are given to the conscientious worker.

In No. 6 of the "Communications" are treated the laws of compression. Besides discussing the older works of the French and English, his own experiments are given. In other numbers, 2, 3, 13, 20, and 21, the properties of metals, the law of the resisting power of iron and stone columns in fire (numbers 12 and 15), and the methods of testing to determine the mechanical properties of wood (numbers 9 and 16), etc., are discussed. The change of the elastic limit and strength of iron and steel due to stretching and crushing is especially to be noticed (number 13).

The "Conventions for the Agreement as to the Methods of Testing Building and Construction Material," of which he was president, were entirely due to his energetic action. The reports of these meetings are contained in numbers 14 and 22 of his "Communications," the latter of which he did not live to complete. These conventions led directly to the permanent International Association which has since been organized.

His work was recognized in all parts of the world. He was made member of the Royal Prussian Academy of Architects, also of the Royal Bavarian Academy of Science in Munich and of the Imperial Academy of Naturalists at Halle; also honorary member of the American Society of Mechanical Engineers, of the Royal Imperial Technological Industrial Museum in Vienna, and of the Royal Bavarian Industrial Museum in Nürnburg, etc., etc.

He died on the 25th of November, 1893, after having given the scientific world the results of his experiments, his theories, and his improvements in testing-machinery, which will ever stand as an indestructible monument to his memory.

APPENDIX B.

STUDY OF IRON AND STEEL BY MICROGRAPHIC ANALYSIS.

By Prof. J. O. ARNOLD, Sheffield, Eng.

I. POPULAR.*

THE ever-increasing severity of engineers' specifications, framed to secure trustworthy qualities in metals used for structural purposes, has undoubtedly had the effect of stimulating metallurgists to make closer scientific investigations, having for their object the determination of the fundamental laws governing the chemical physics of metallic alloys, and the exact working of those laws with reference to the ultimate mechanical properties of metals. More particularly in connection with iron and steel, the fact is now generally recognized that materials identical in chemical composition may possess widely different mechanical properties. When the observations of the analytical chemist, although indispensable, nevertheless became of more limited value, the metallurgical physicist appeared on the scene and, it must be confessed, very ably put forward a theory that the otherwise inexplicable differences in the practical properties of chemically identical masses of iron or steel must be due to allotropic changes in the iron itself, and, for a time, allotropic molecules became fashionable. When, however, practical metallurgists found that the allotropic school put forward as part of their belief the startling creed that chromium and tungsten, silicon, sulphur, and phosphorus soften steel, it became evident that, as far as the practical applications were concerned, there was a rift in the theoretical lute, and that the indication, furnished by some other line of research were destined to explain the puzzling effects frequently observed.

During the last few years it has become more and more apparent that the veil would only be lifted from the mysteries of metals by micrographic analysis—a fact of peculiar interest and encouragement to young experimenters starting along the thorny path of research. Young scientists, after executing and publishing patient and valuable work, will often find that their efforts are received with indifference. Such investigators should remember that the micrographic analysis of iron and steel was inaugurated thirty-five years ago by Dr. Sorby, in a research which, for patience of execution and sterling value of results achieved, has seldom been excelled. The sagacity of Dr. Sorby's preconceived idea, that metals should be regarded as crystallized igneous rocks, is now generally recognized. Nevertheless practical metallurgists have only quite recently realized that Sorby's research founded the science of metallography. This science is destined in the near future to become an indispensable adjunct to chemical analysis, and it has already practically proved that the metallurgical engineer, instead of groping for the causes of abnormal mechanical effects in the outer darkness of molecular metaphysics, may often readily find such causes well within the range of actual vision by means of a cheap microscope.

In a word, the venue of trial has been changed from molecules to crystals, or, to be more strictly accurate, from molecular to intercrystalline cohesion, or, it may be, adhesion. The magnitude of this change is hardly capable of mental realization, but its enormity may be vaguely grasped by recalling Lord Kelvin's calculation of

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* *From Iron and Coal Trades Review*, 1896.

the probable approximate dimensions of a molecule, namely, that if a single drop of water be magnified up to the size of the earth, the constituent molecules of the drop would be somewhere about the size of marbles. It is encouraging to know that the growing importance of micrographic analysis has become a matter of international recognition, and among patient investigators engaged in its development in America, England, France, Germany, and Holland may be mentioned the names, respectively, of Stead, Osmond, Martens, and Behrens. There is little doubt that the efforts of these and other workers will soon raise metallography to the rank of a definite science, but the path of the student seems likely to be rendered necessarily difficult by the assignation of names to apocryphal constituents, and the curse of synonyms already hovers over the science.

Many people are inclined to associate the microscopic examination of steel with a grave peering into fractures with a hand lens. It is well to clearly understand once and for all that the fracture of a piece of steel or iron has but little correlation with its ultimate structure. The latter is ascertained, firstly, by obtaining a perfectly polished section of the metal; secondly, by delicately etching with acid the prepared surface, in order to reveal the constituents, just as the structure of a macadamized road is revealed after a heavy flush of rain. The results so far obtained by this method of examination have proved that even a chemically pure metal is not a homogeneous solid, being built up of a number of primary metallic crystals, which may themselves break up into a large number of secondary crystals. It is extremely probable that the mechanical properties of such a metal are measured, not by molecular cohesion—not even by the cohesion between the secondary crystals—but by the attractive force acting between the facets and the large primary crystals.

Passing on to the question of the so-called alloys, the indications of the microscope have already gone far to negative the generally accepted idea that an alloy consists of a homogeneous solution of one metal in another, or of a non-metallic element in a metal. To take the specific case of steel in its ordinary state, as used for structural purposes, the microscope has forever removed from the mind of the engineer any idea that he has to deal with a homogeneous mass. To bring steel into its purest and essential form, in which iron and carbon are the main constituents, the microscope has proved that steel is grown from iron by gradually increasing the carbon present, and that it reaches maturity, otherwise a comparatively homogeneous mass of true steel, when the percentage of carbon approximates 0.9 per cent. This percentage constitutes the critical microscopical point of steel, and has been named the "saturation-point." The addition of more carbon produces a supersaturated steel, slowly progressing towards pig iron. However, structural engineers are more immediately concerned with unsaturated steels—that is to say, steel containing less than 0 9 per cent of carbon. In such material, what may be called a semi-critical point of great importance, from an engineer's view, is presented in iron containing 0.45 per cent of carbon. The material then consists of an intimate mixture of perfectly distinct crystals of iron, and of true steel containing 0.9 per cent of carbon, in equal proportions. This metal presents mechanical properties intermediate between those of pure iron and true steel. Should an engineer in his specification demand a carbon higher than 0.45 per cent, he will obtain a material possessing the characteristics of steel rather than those of iron. On the other hand, in metals containing less than 0.45 per cent of carbon the characteristics of iron will predominate. The above statements have reference to an ideal case, the consideration of which is, however, absolutely necessary to form the base-line of steel metallurgy. In practice the case becomes complicated because of the presence in structural steels of from 0.5 per cent to 1 per cent of manganese. The influence of the quantity last named on the mechanical properties of steel is well known, and its effect on the microscopic structure is remarkable. Nevertheless, it seems to have escaped the observation of most steel microscopists. There is little doubt that the observed effects are due to the formation of a remarkable triple compound of iron, manganese and carbon.

Our knowledge of this subject is far from complete; it is, in fact, a branch of the subject requiring immediate and rigorous investigation. It is, however, certain that in iron containing 1 per cent of manganese the microscopic saturation-point

marking the conversion of the iron into a homogeneous compound, steel, appears about 0.65 per cent of carbon. We have in this fact a satisfactory explanation of the well-known mechanical differences between Swedish and English Bessemer spring-steel of like carbon, the kinder properties of the Swedish material being due to the fact that it contains only about ¼ per cent of manganese. Further investigations, not yet ripe for publication, have gone far to indicate that the specific action of the elements nickel, chromium, tungsten, and silicon are due, not to the elements *per se*, but to a remarkable series of double carbides of the respective elements with iron. The substances above mentioned are, however, often useful when employed to obtain the mechanical properties demanded by unusually severe specifications. But the steel metallurgist is confronted by the invariable presence of two elements, the action of which is always injurious, frequently to an extent seemingly out of all proportion to the percentages present. Speaking broadly, sulphur is the more deadly enemy, for reasons which open up a wide field of research in general metallurgy. The cause of the more injurious action of sulphur may be stated in a word. Sulphide of iron, which (and not sulphur) is the substance with which the engineer has to reckon, is far more fusible than phosphide of iron. The extent to which mass fragility may be produced by very small quantities of a fluid or semi-fluid constituent, after the main mass of the material has solidified, is hardly capable of exaggeration, and microscopical evidence will presently be published which will conclusively prove that proportions of sulphur hitherto deemed harmless may, under certain conditions, produce a remarkable mass weakness fully capable of accounting for mysterious and disastrous effects. Sulphide of iron is incapable of adherence to the constituents adjacent to it within the mass, so that the extent of its injurious effects will depend upon the form in which it exists mechanically. Its least injurious form is that of fused globules, which are practically equivalent to minute blowholes. Its most dangerous form is that of attenuated membranes enveloping groups of crystals, and forming long lines of weakness equivalent to minute cracks. The mechanical distribution of the semi-fluid sulphide during the rolling and hammering of steel presents dangerous possibilities and requires rigorous microscopical investigation.

A fruitful field of research which has already yielded important results is the microscopical determination of the changes taking place during annealing. The constituents chiefly involved in this change are the carbides and sulphides, and the results already obtained have completely negatived the accuracy of the generally accepted theory of annealing. It is not intended in the present article to anticipate, by premature publication, the remarkable influence of annealing on the distribution of sulphide of iron, and the increase in mechanical strength following such redistribution. With reference to carbides, however, it may be pointed out that in one of our leading text-books on metallurgy the toughening influence of the process of annealing is attributed to three causes: 1. A change of hardening carbon into carbide carbon. 2. A breaking up of large crystals into minute crystals. 3. A distribution of carbide carbon from crystalline pellets into finely-diffused particles. Micrographic analysis has shown that not only are the foregoing statements inaccurate, but they are also opposed to fact, for the following reasons: 1. There is no hardening carbon in steel castings. 2. The crystals become much larger on annealing. 3. The carbide carbon is entirely concentrated into crystalline pellets. The above case is a single example of the light destined to be thrown on the metallurgy of steel by micrographic analysis. It, however, yet remains for engineers to fully grasp the realities briefly set forth in this article. To do so it is necessary to examine a comprehensive collection of properly prepared iron and steel micro-sections.* The recognition of the value of the science to metallurgical engineers has undoubtedly been retarded by a pedantic adherence to the reproduction of the structures observed by photography.

It seems that the idea that the camera is the George Washington of inanimate life has not yet been exploded. As a matter of fact, there is only one philosophical instrument capable of conveying more inaccurate impressions than the camera, and that is the gas-meter. Any one who is familiar with the actual structures of steel,

* See Plates IX and X.

and with the foggy series of photographs published from time to time to represent them, will admit the accuracy of the above statement. The technical difficulties involved in photographically reproducing the micro-structure of opaque objects under high powers are so great, that at present the only reliable means of reproduction is laborious hand-drawing, employing either a micrometer or camera lucida; and the only reasonable objection to such a course is an imputation of *mala fides* to the operator.

For obvious reasons, the structure of iron and steel has herein received most attention; micrographic analysis is capable, however, of application to many alloys, and of explaining not only their mechanical properties, but also their electrical conductivities, so that the science is of importance, not only in practical metallurgy, but also in theoretical physics, based on observations of the electrical properties of alloys.

II. TECHNICAL.*

The study of this very important branch of steel metallurgy, initiated thirty years ago by Dr. Sorby, has attracted considerable attention on the Continent. In Germany, particularly, great strides have been made in its development under the superintendence of Professor Martens. By English metallurgists it has been until quite recently almost neglected, or condemned with faint praise. The mechanical difficulties of preparing a perfect section and of delicately etching the surface, so as to reveal its true structure when examined with high powers as an opaque object, are considerable. The author has only overcome these obstacles after some years of laborious experiment, for which, however, he has been amply repaid by the discovery that the laws determining the structure of iron containing various percentages of carbon are fixed and concordant for given physical conditions; in fact, from puzzling chaos he has been enabled to evolve order of a most interesting character, supplying, moreover, the key to the position he was attacking.

The Constituents of Iron and Carbon Steels.

Pure Iron.—Perfectly pure iron is never met with in commercial masses, but in Swedish Lancashire hearth-rolled bars, containing in their average analysis 99.8 per cent of iron, groups of almost chemically pure crystals of the metal may be met with. They are readily distinguished by their well-defined facets and angles, and by the fact that they remain bright and smooth even after prolonged attacks by the excessively dilute nitric acid used for etching, which merely penetrates and makes visible the fine junction-lines of the crystals. In Fig. 636 is shown a micrometric reproduction of crystals of pure iron viewed by direct illumination and magnified 600 diameters. Their geometric form agrees most nearly with that produced by interfering cubes and octahedra with dominant cubic faces.† It is, however, unusual to meet with such well-defined and geometrical crystals as those figured, because of the distortion-stresses taking place in the metal during cooling after crystallization, which phenomenon the author's experiments ‡ indicate as occurring at a moderate red heat, the formation commencing at 750° C. and being completed at 720° C.

Slightly Impure Iron.—In wrought iron and in mild steels the free iron crystals are often somewhat contaminated with a little residual carbon, which causes them during the process of etching to assume a pale-brown tint and a rough surface. The amount of carbon so involved is very small, seldom exceeding 0.05 per cent, and its mechanical influence is insensible. The author, therefore, will not at present further discriminate between the two kinds of iron crystals, though the exact nature of the carbide existing in the tinted crystals has some molecular interest in connection with Osmond's point A_R 3. When very mild steels are submitted to prolonged heating in a vacuum at a temperature of 1400° C. and are then cooled in air, the microstructure of the steels undergoes a distinct change, in which the knots of

* From *Inst. Civ. Engrs.*, vol. CXXIII. (1896), p. 137 *et seq.*

† In a recent communication to the Royal Society Mr. Thomas Andrews, F.R.S., has shown that in heavy slowly-cooling masses of wrought iron the large primary crystals often split into numerous secondary cubes.

‡ *Journal of the Iron and Steel Institute*, 1894. No. 1, p. 132 *et seq.*

normal carbide of iron disappear and a large increase takes place in the number of tinted crystals. These facts are correlated thermally with a large permanent increase in the heat evolved at A_R 3 on cooling, the carbon change-point almost disappearing at A_R 1 and its position at A_R 3 being raised about 10° C. These results are consis-

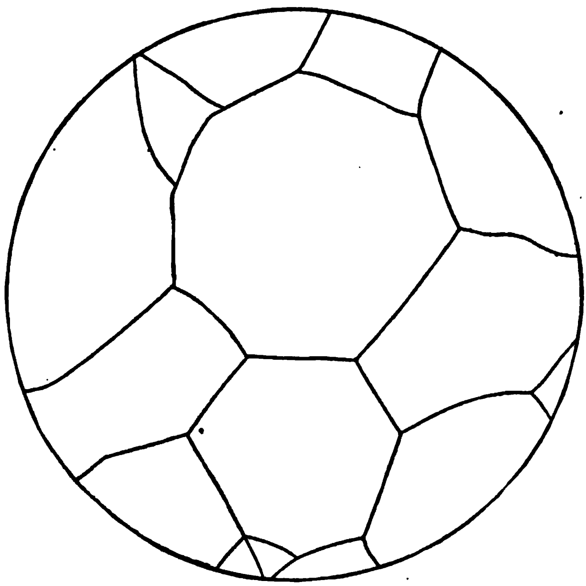

Fig. 686.—Pure Iron Crystals Magnified 600 diameters.

tent with the theory that there may exist traces of a carbide of iron intermediate in formula between the normal carbide Fe_3C, and the all-important subcarbide to be presently described.

*Diffused Normal Carbide.**—These areas, when the polished section is immersed in very dilute nitric acid, are at once partly covered with a dark-brown film of carbonaceous coloring matter, the latter thus constituting an invaluable automatic staining medium. The dark areas, as will be shown subsequently, consist of iron containing about 13 per cent of normal carbide, Fe_3C, diffused through its substance in the form of small, ill-defined plates and granules. They also mark the preliminary stage of formation of the "pearly constituent."

The Pearly Constituent.—This constituent is best developed in annealed steels, and presents the well-known hard and soft laminæ discovered by Dr. Sorby. It has already been shown that the hard laminæ are crystals of Fe_3C. The soft interspaces are nearly pure iron. The parallel carbide plates may be wavy or straight, and they differ much in thickness and their distances apart. When the iron interspaces are very wide, the carbide plates are distinctly seen to be in relief, the fibres of the polishing blocks having excavated the soft iron. Sections containing much "pearly constituent" present on etching a beautiful play of pearly or opaline interference colors, which, if the etching is very light, are permanent.

Crystallized Normal Carbide (Fe_3C).—This substance, exclusive of its occurrence in the pearly constituent, may gather into large sectional rivers or into isolated masses. It requires then an experienced eye to distinguish it from perfectly pure iron, but, as a rule, the fact that it is always in relief and its brilliant silvery surface serve to identify it.

Graphite.—This is Ledebur's "temper-carbon." For English-speaking metallur-

* These correspond with the "amorphous iron" of Dr. Müller.

gists a more unfortunate name could hardly have been chosen. "Annealing carbon" would have been better; but to make its nature quite clear the author will throughout this paper employ the name graphite, as expressing for all practical purposes what the substance is. In steel it occurs in the form of dark rounded dots (or more rarely in short, worm-like masses) well defined against a background of pale iron.

Subcarbide of Iron (or β iron).—But one more constituent remains to be described, and this, if Mr. Osmond's theory be true, must be β iron charged with dissolved carbon. On lightly etching a polished section consisting mainly of this compound it retains its polish but assumes a "black-leaded" appearance, due to a very faint coating of dark carbonaceous matter. It seems homogeneous and apparently non-crystalline, but probably consists of minute crystals, the junction-lines of which are beyond the range of microscopic vision or are obscured by the faint carbonaceous deposit. When deeply etched, this substance becomes covered with a velvet-black deposit, which may be removed by the finger, staining the latter. The body just described is found only in hardened or hardened-and-tempered steel. The author will presently produce whats eems to him conclusive microscopical thermal and magnetic evidence that this substance is not an allotropic modification of iron, but a definite though remarkably attenuated and unstable carbide of iron of intense hardness, and corresponding with the formula $Fe_{24}C$.

Details of Microscopic Observations.

The structures illustrated, Plates VII and VIII, were all drawn from the microscope, when necessary a micrometer being used on correspondingly graduated circles 28 inches in diameter. The drawings were then reduced by photography to the diameter of the microscopic field. The labor involved in carrying out this process was great, but the results depict the structures with an accuracy unattainable by direct photography. Direct illumination was employed throughout.

Normal Steel No. 1, Pl. VII (Carbon 0.08 per cent).—The structure consists of irregular crystals of iron, amongst which are sparingly distributed small dark knots of the diffused normal carbide areas. On comparing this section with that of the pure iron, it will be seen that the presence of even 0.08 per cent of carbon is at once decisively revealed by the microscope.

Annealed Steel No. 1 † (Carbon 0.08 per cent).—The effect of annealing had been to produce a distinct increase in the size and geometry of the iron crystals, and to gather the Fe_3C, diffused through the dark normal areas, into isolated patches of coarse pearly constituent surrounded by thick sectional meshes of crystallized Fe_3C.

Normal and Annealed Steels, No. 1½ (Carbon 0.21 per cent).—These sections were in all respects intermediate between those of steels Nos. 1 and 2. It was not therefore deemed necessary to illustrate them.

Normal Steel, No. 2, Pl. VII (Carbon 0.38 per cent).—This section consists of a mixture of crystals of iron, and large irregular dark areas of diffused normal carbide, the latter occupying on an average nearly half the field.

Annealed Steel, No. 2 Pl. VII (Carbon 0.38 per cent).—On annealing, the iron crystals have become larger and more definite, whilst the dark areas have aggregated; and on cooling the components have segregated, forming striæ of crystallized Fe_3C, divided by spaces of iron. The groups of striæ are often partly surrounded by sectional meshes of Fe_3C, a few isolated striæ of which compound may be sometimes observed between the junctions of the iron crystals.

Annealed Steel, No. 2 † (Carbon 0.38 per cent).—This section gives a general view of the structure over a comparatively large area. It forms a beautiful microscopic object resembling an irregular mosaic pavement composed of white crystals of iron and large irregular patches of the pearly constituent, showing splendid interference colors. Of course a magnification of 100 times a linear dimension is insufficient to resolve the striæ of the pearly constituent.

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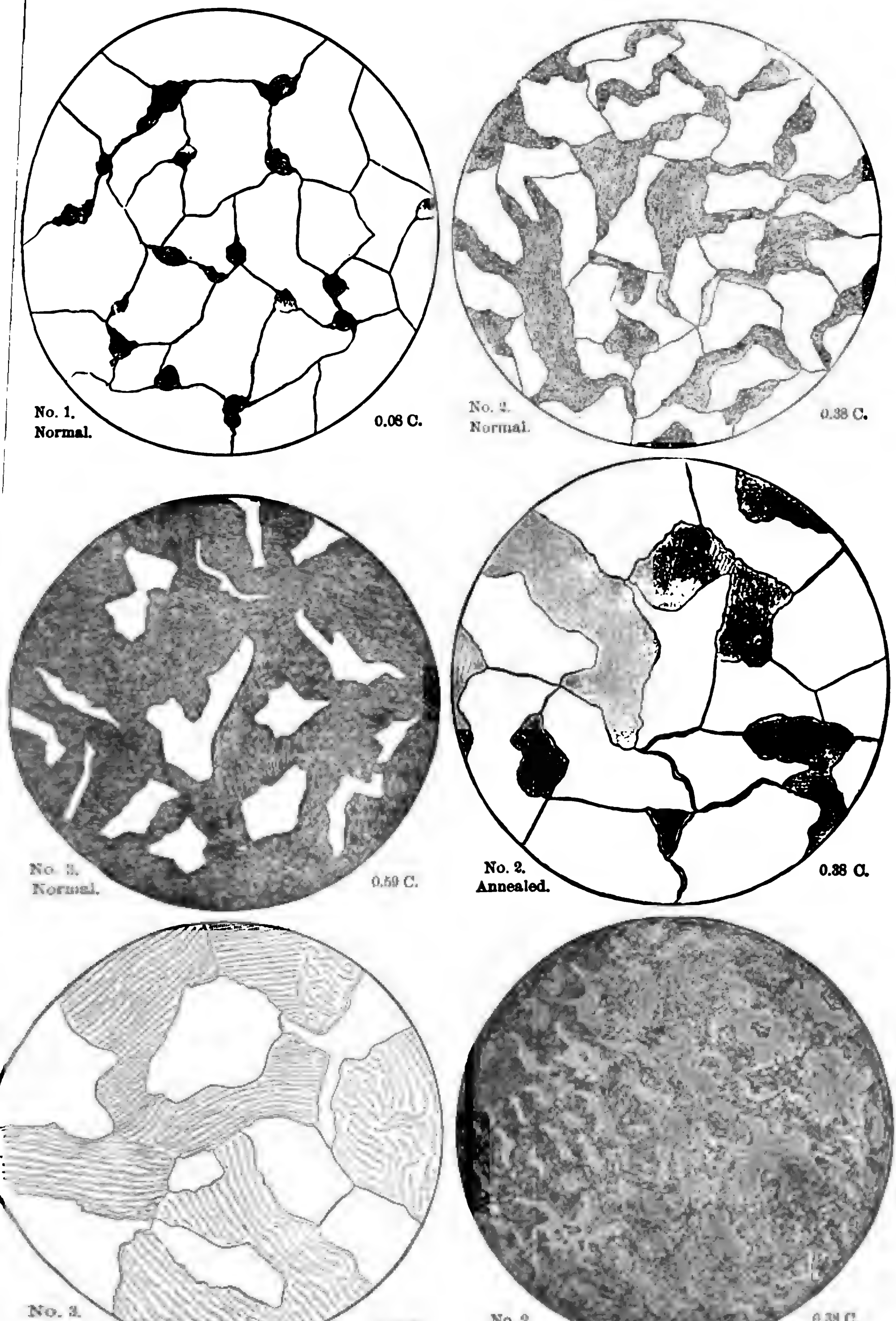
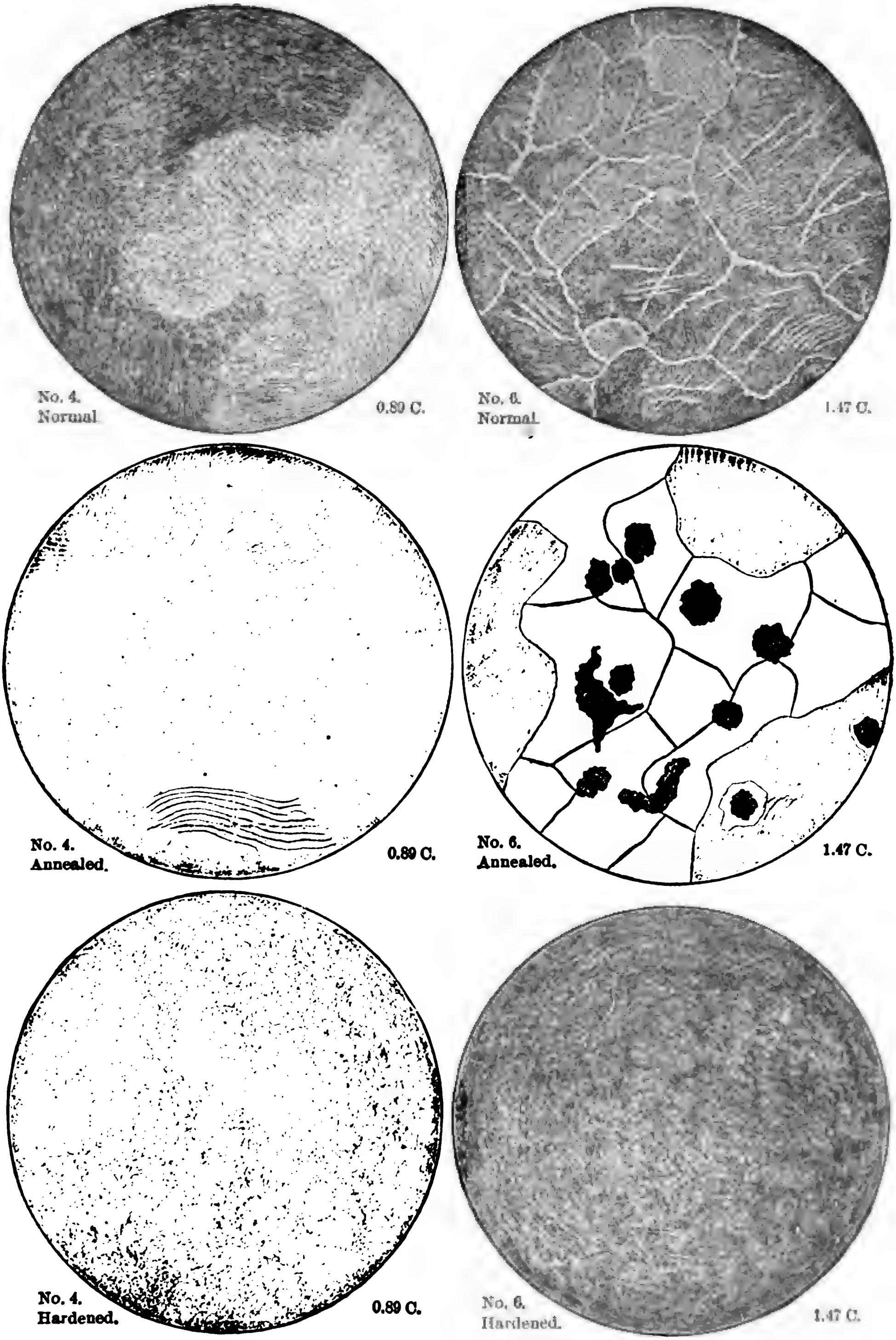
* In examining the engravings of the sections a lens will be found useful for some of the finer structures.

† This figure is given in the author's plate, but was not reproduced for this work.—J. B. J.

PLATE VII.



icroscopic Sections of Steel, magnified 500 diameters. Treatment and per cent carbon indicated.
Drawn by Prof. J. O. Arnold. (*Inst. Civ. Engrs.*, vol. cxxiii. (1896) I l. 4.)



Microscopic Sections of Steel, magnified 500 diameters. Treatment and per cent carbon indicated. Drawn by Prof. J. O. Arnold. (*Inst. Civ. Engrs.*, vol. CXXIII. (1896) Pl. 4.)

The General Influence of Annealing on Mild Cast Steel.

As No. 2 steel contains about the same percentage of carbon as that contained in high-class castings, it will be well to discuss in connection with it the general principles underlying the operation of flame annealing. The surface oxidation of carbon resulting from this process will be neglected, as its effect is comparatively small, and only the action of annealing on the main portion of the steel which is unaltered in ultimate chemical composition will be considered.

The ideas prevalent among metallurgists on this subject are often very erroneous. It has been stated, both in text-books and in practical papers, that the action of annealing is to produce smaller crystals. As a matter of fact, the crystals of an annealed steel are always larger than those existing in the metal before annealing. The idea of smaller crystals has doubtless arisen from a confusion of effect with cause. After annealing, the crystals of the fractured metal do appear to the eye smaller than those in the original material; but the reason for this is found in the fact that during the process of rupture they elongate, what are really seen being the ends of the ductile, and consequently drawn out, crystals, and not (as in the case of the comparatively brittle unannealed metal) the originally existing facets. Also, frequently, what are regarded as crystals in the fracture of a brittle unannealed steel casting are really groups containing many crystals, originally bounded by lines of great intercrystalline weakness, along which rupture has naturally taken place. As a rule, the fracture of steel bears little or no relation to its true micro-structure. It has also been stated that on annealing, the iron and the pearly constituent become more intimately mixed ; for which Dr. Sorby has been quoted as the authority. Dr. Sorby's general conclusion on this matter was accurate, and was exactly opposite to that attributed to him. The mistake seems to have arisen from an imperfect knowledge of the meaning of the word "segregate." The true action of annealing on a moderately mild steel, containing, say, 0.35 per cent of carbon, is as follows: 1. The comparatively small and distorted crystals of the original metal become larger and more geometrical in form (they are therefore freer from internal stresses), and the intercrystalline cohesion, if originally weak, is much strengthened.* 2. The carbonized areas existing in the unannealed steel, chiefly in irregular elongated masses, gather together during the slow cooling into rounded or harp-shaped areas, in which form they favor the continuity of the iron crystals to a much greater extent than the original arrangement. 3. The rounded or harp-shaped areas, into which are concentrated the normal carbide of iron split up during the slow cooling into plates of crystallized Fe_3C, separated by large interspaces of iron; hence the latter become dovetailed into the main body of the iron crystals. This continuity, however, is not perfect, being frequently broken by the sectional meshes partly environing the laminated areas. Thus long lines formed by a juxtaposition of two distinct constituents are broken up, and the iron becomes almost continuous throughout. In fact, the carbon is concentrated into small plates suspended in the iron, mixed with only about 5 per cent of the total metal instead of being distributed in large more or less continuous areas forming about 40 per cent of the mass. The foregoing statements are common to the cases of forged steel, as well as of the metal as cast, but they apply with peculiar force to the last-mentioned material, in which the intercrystalline cohesion is usually weak; which is not often the case in forged steels, because the work put upon the material has already repaired the faulty crystal-junctions in a manner analogous to the influence exercised by annealing.†

In order to render the above facts more clear, half-fields of No. 2 steel in the annealed and normal conditions are exemplified in Fig. 14.‡ The sections from which these were drawn were very lightly etched so as to bring out only the carbonized constituents without developing the lines marking the intercrystalline junc-

* Perfect intercrystalline cohesion is synonymous with that hitherto mysterious essence known to the practical man as "body."

† The question of the influence of annealing on the various types of steel castings is of sufficient importance to warrant its special consideration in a separate paper, for which the author has for some time past been collecting data, some of which are of a startling nature.

‡ Not reproduced here.

tions. The several sections of No. 2 steel show clearly that existing views as to steel being built up of a series of cemented cells are erroneous. As already stated, streaks of carbide cement may now and then be seen between the junctions of the iron crystals, but such constitute incidents, and not a principle. As a matter of fact, if the facets of the iron crystals were really united by Fe_3C cement, a mild steel would be easily fractured by a blow from a heavy sledge-hammer. The author, from the results of many experiments, confidently makes the following statement:

If the cohesive force acting between the facets of crystals is from any chemical, thermal, or mechanical cause seriously weakened, the metal will appear to be very brittle, owing to rupture under the influence of a sudden shock occurring along the weak junction-lines, in spite of the fact that the molecular cohesion may be perfect and the individual crystals ductile.*

From the foregoing statement it will be obvious that a metal may be soft to the drill or under compression, and yet brittle under impact, exhibiting little or no ductility in tension. It is also clear that in such a case chemical analysis is useless. The author, however, is not yet prepared to state the exact means by which intercrystalline weakness may be measured by the microscope, but he is hopeful that in the near future such measurements may be possible. That in nearly pure iron the intercrystalline cohesion and the molecular cohesion may be equal, is proved by the tensile test of No. 1 steel annealed. This metal is composed of large definite crystals, yet the elongation was 53 per cent and the reduction of area at the point of fracture in tension was 77 per cent.

Normal Steel No. 3, Pl. VII (Carbon 0.59 per cent).—In this section the dark normal carbide areas considerably exceed the now isolated and highly distorted crystals of iron.

Annealed Steel No. 3, Pl. VII (Carbon 0.59 per cent).—This section confirms, and presents on a larger scale than No. 2 steel annealed, the breaking up of the dark areas into striæ of crystallized Fe_3C separated by interspaces of iron.

Steel No. 3½ (Carbon 0.74 per cent). Slightly annealed.—This section was in all respects intermediate to the normal sections of steels Nos. 3 and 4, containing fewer iron patches than the former.

Normal Steel No. 4, Pl. VIII (Carbon 0.89 per cent).—This section presents a feature of vital importance in connection with the theory of steel which the author will presently enunciate. The entire structure consists of ill-defined crystals forming dark areas of iron containing suspended normal carbide, whilst crystals of iron free from suspended carbide have necessarily altogether disappeared. In other words, iron containing 0.89 per cent of carbon presents a critical microscopical point which will be hereinafter referred to as the "saturation-point"; steels in which the carbon falls below 0.89 per cent will be termed "unsaturated"; whilst steels containing more than 0.89 per cent carbon will be distinguished as "supersaturated," for reasons to be presently stated.

Annealed Steel No. 4, Pl. VIII (Carbon 0.89 per cent).—This section consists entirely of crystals of the pearly constituent. The crystallized striæ of Fe_3C are in nearly parallel lines, some straight, others wavy. Small isolated masses of this compound also occur sparingly. The thickness of the plates and of the iron interspaces vary considerably. In one area the hard plates are in such relief owing to the wearing away of the broad, soft iron interspaces, that they actually cast microscopic shadows, as indicated in the figure. The microsection of this steel, consisting entirely of the pearly constituent, presents to the eye a beautiful play of colors resembling those of mother-of-pearl.

Normal Steel No. 5,† (Carbon 1.2 per cent).—The main portion of this section is similar to that of normal steel No. 4, but each crystal or group of crystals is

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* Purely scientific investigators dealing with the physics of iron discourse freely on molecules and their distances, but they ignore crystals and their comparatively huge interspaces. This is a grave error; e.g., there is little doubt that magnetic properties are much influenced by the dimensions of the crystals into which the molecules are grouped. The author emphatically reiterates that deductions explaining observed mechanical facts on the basis of allotropic changes in the molecular architectures of metals are valueless, unless the effects due to crystalline architecture have been previously determined and allowed for. The effects due to the first-mentioned cause are often very small in comparison with the effects produced by intercrystalline causes.

† Not reproduced here.

surrounded by a sectional mesh of Fe_3C. Isolated striæ of the latter compound also occur. It must be remembered that the strings of carbide which appear sectionally as a coarse and irregular network are in reality, when translated into the solid, more or less perfect investing membranes. This statement is proved by the fact that both transverse and longitudinal sections present the same characteristics.

Annealed Steel No. 5,* (Combined Carbon 0.92 per cent, Graphite 0.28 per cent).—This section possesses features of special interest. It presents two distinct types of field, which are reproduced in two half-fields. In one will be observed crystals or groups of crystals composed of the pearly constituent enclosed in very large sectional meshes of Fe_3C. These thick membranes have evidently resulted from a confluence, during the slow cooling, of the comparatively small membranes present in the normal steel. In parts of the section from which meshes are absent there are, however, round almost black patches of graphite, set for the most part in the centres of round patches of bright iron, the remainder of such field being as usual composed of the pearly constituent. It would therefore appear that when the mobilized masses of carbide attain a certain magnitude, they act during the slow cooling like very highly carbonized white pig-iron, dissociating into nearly pure iron and graphite. The temperature at which this separation takes place will be considered in connection with the annealed sample of No. 6 steel.

Normal Steel No. 6, Pl. VIII (Carbon 1.47 per cent).—In this section the dark background of iron permeated with diffused Fe_3C is much broken up by thick irregular meshes of crystallized Fe_3C. The enclosed crystals also contain large fern-like streaks of the crystallized carbide, the whole constituting a beautiful and striking microscopic object.

Annealed Steel No. 6, Pl. VIII (Combined Carbon 0.33 per cent, Graphite 1.14 per cent).—In this section about one third of the area consists of the pearly constituent, the other two thirds being composed of iron crystals largely spotted with dark round patches, and short worm-like masses of graphite. The latter must have separated below the temperature of the carbon change point $A_R 1$, which is about 685° C., because the large masses of carbide described in connection with annealed steel No. 5 would in the present case be still greater; and not only have they become totally decomposed, but have evidently also gathered in much of the carbide existing as small plates in the pearly constituent. Hence, as the plates so collected would not have crystallized till the temperature had fallen to about 680° C., it appears certain that the decomposition of the Fe_3C into iron and graphite must take place at or below $A_R 1$ i.e., at a low red heat. The interesting fact that this dissociation is facilitated by pressure is proved by the investigations of Mr. B. W. Winder, who found that hard file-steel leaving the rolls at a low red heat almost invariably contained graphite in large quantities, whilst similar steel finished at a fair red heat was almost devoid of free carbon.

Annealed Steel No. 6,* (Combined Carbon 0.33 per cent, Graphite 1.14 per cent).—This section presents a large area of the graphitic metal, showing the crystals of iron, the spots of graphite, and the curiously irregular masses of the pearly constituent in which is contained the 0.33 per cent of combined carbon present. At 100 diameters the microscope is of course incapable of resolving the striæ of the pearly constituent, which, however, presents a play of gorgeous colors. From the three graphitic sections referred to, it will be seen that supersaturated steels are always very liable to deposit graphite on annealing. Such a phenomenon is seldom or never observed on or below the saturation-point.

The foregoing microscopical facts have been known to the author for about three years, having been ascertained by an examination of another series of steels similar to those now under consideration. But as it was unexpectedly found that the harder steels contained about 0.3 per cent of manganese, the author rejected the first series, and made a purer set of steels upon which to commence the research afresh. As the result proved, the comparatively high manganese in the hard steels did not seriously affect the results just described, and the first series now constitute confirmatory evidence, which has also been augmented by the examination of many samples of commercial steels.

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* Not reproduced.

General Theory Based on the Microscopical Results.

The evidence given by the microstructure of the steels is thus interpreted by the author:

1. The sharply defined localization of the areas containing respectively the diffused normal and crystallized carbides (until the saturation-point at 0.89 per cent of carbon is reached) seems to confirm beyond doubt the accuracy of the general conclusion of Dr. Sorby—that at high temperatures a compound of iron and carbon exists, which on cooling splits into iron and an intensely hard compound very rich in carbon.

2. The fact that the dark carbonized areas of normal steels increase proportionally to the carbon until the saturation-point is reached seems quite incompatible with the theory that at a high temperature the carbon is in a free state in mere solution. Under such conditions the carbide of iron would be evenly diffused after its deposition *in sitû* on cooling, and would on etching yield an almost homogeneous microscopic field, darkening in color as the carbon increased; inasmuch as the stronger the solution at high temperatures, the greater the amount of diffused carbide in the cold metal, and, *cæteris paribus*, the thicker the deposit of carbonaceous coloring matter released on the surface of the etched section.

If it be admitted that the dark areas in normal steels and the striated areas in the corresponding annealed metals are mixtures resulting from the decomposition of a compound existing at temperatures above the change-point $A_R 1$, it necessarily follows that at the saturation-point (at 0.89 per cent of carbon) the whole mass of the iron is at a full red heat in combination with the carbon, and hence that the percentage of carbon in the saturated steel is also the percentage of carbon in the formula of the compound. Therefore the compound will contain 0.89 per cent of carbon and 99.11 per cent of iron, corresponding with the formula $Fe_{24}C$, which requires 0.884 per cent of carbon.

In the case of a supersaturated steel made by gradually adding, say 1.5, per cent of carbon to pure iron in a molten state: when the iron has combined with 0.89 per cent of carbon it will have been converted into a carbide of formula $Fe_{24}C$; but on adding more carbon a portion of the subcarbide will be carbonized to the normal carbide Fe_3C; thus·

$$Fe_{24}C + 7C = 8Fe_3C.$$

The molten mass then consists of a mixture of the normal carbide with subcarbide of iron. On cooling, the subcarbide decomposes into ill-defined crystals of iron permeated with diffused Fe_3C, whilst the surplus normal carbide is thrown off in the form of membranes enveloping the irregular crystals of the mixture resulting from the decomposition of the subcarbide.

The Structure of Hardened Steels.

To obtain more conclusive microscopic evidence of the accuracy of the theory just enunciated, it was obviously necessary to determine the structure of hardened steel below, on, and above the saturation-point. When it is remembered that even the skill of Dr. Sorby was baffled in all his efforts to obtain satisfactory sections of hardened steel, it is not surprising that, although possessing superior appliances, the author's experiments in this direction were for several years almost fruitless, yielding most puzzling and erratic results. However, comparatively recently, the author possessed himself of the key to the position, in the fact that it was absolutely necessary to harden the samples from a nearly white heat, without allowing them to come into contact with either air or water. This was because the decarbonizing action of a film of magnetic oxide on the surface of a piece at a full red heat extended irregularly to such a depth that it was almost impossible in the flint hard steels to grind off the partially decarbonized surfaces without disturbing the structure or "letting down" the steel. This fatal defect was finally removed by the following simple though somewhat costly plan. Each microsection was polished and encased air-tight in thin plates of the same steel in the manner indicated in Fig. 637. The encased

microsection was then slowly heated to about 1050° C., and was quenched with the greatest possible rapidity in a large tank of ice-cold water. On drying and removing the casings the section, although it had been heated during half an hour up to an

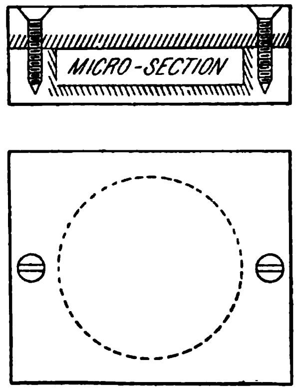


FIG. 637.—Twice full size.

incipient white heat, was found to be quite bright and absolutely unoxidized on the lower polished face. On lightly etching three typical sections the following results were obtained:

Hardened Steel, *No.* 2, Pl. VII (Carbon 0.38 per cent).—On being etched, the sample assumed a roughish texture and a dull, somewhat dark-gray tint. On lightly removing the gray deposit the steel was found to consist of two distinct constituents, viz., free iron and an amorphous substance to which the acid had communicated a dark color. In some fields the iron, and in others the dark constituent, predominated. The author affirms the latter to be subcarbide of iron. The section figured represents an average field. The shock of the sudden cooling seems to have dispersed the iron through the dark substance in masses irregular in size and fantastic in shape—many particles, no doubt, being too small for separate microscopic definition.

Hardened Steel, *No.* 4, Pl. VIII (Carbon 0.89 per cent).—This section, on being very lightly etched, retained its polish, but assumed a "blackleaded" appearance. When examined under the microscope the field at first sight presented a brownish-colored blank, in which no crystalline structure could be detected. A prolonged and careful examination showed that the section really possessed an indefinite granular roughness, but no crystalline junctions could be detected. It is, however, probable that the mass really consists of minute crystals, the boundary-lines of which are beyond the reach of microscopic vision or are rendered indefinable by the faint carbonaceous deposit. This is the only practically homogeneous section the author has ever obtained during many years of close study of the microstructure of steel and iron.

Hardened Steel, *No.* 6, Pl. VIII (Carbon 1.47 per cent).—On being etched, this section behaved in every respect like hardened steel No. 4. The groundwork of the section was also found to be identical with the saturated metal, but all over it was spread a network of fine meshes, together with isolated striæ and irregular dots of a substance microscopically corresponding in all respects to Fe_3C.

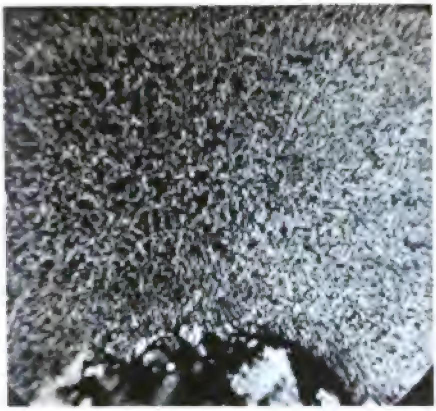
Thus the microstructures of the hardened steels seem in accordance with the author's theory. The unsaturated steel possesses a structure such as might be expected from a suddenly quenched mixture of free iron and subcarbide of iron.* The saturated steel fulfils the necessary theoretical condition of homogeneity, whilst

* The non-homogeneous nature of hardened unsaturated steel is best seen in oil-quenched gun-steel containing about 0.3 per cent of carbon, the almost black subcarbide areas being fantastically enmeshed in free iron.

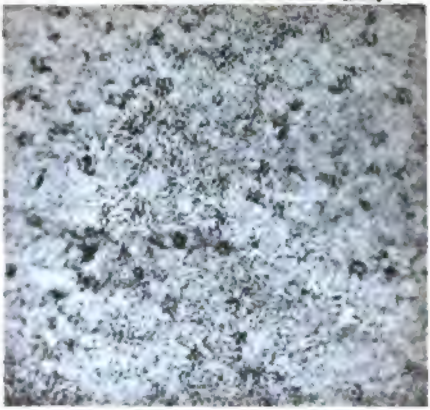
the supersaturated steel decidedly reveals the presence of surplus meshes of normal carbide of iron.

NOTE BY THE AUTHOR.

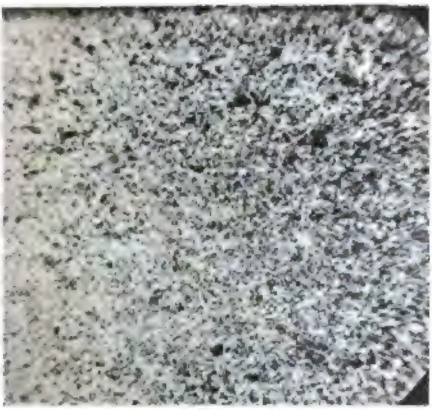
Photomicrographs of Steel.—The microscopic sections shown in Plates VII and VIII are from drawings. Those shown in Plates IX, X, and XI are reproduced photographs, taken directly from the specimen, and they reveal these just as they would appear to the observer, so far as can be done by photography. The drawings indicate the crystalline arrangement much better than the photographs, but reliance must be placed in the competency and faithfulness of the draughtsman, who is of course the observer. With the photographs the personal equation of the observer is eliminated. The two taken together reveal, to some extent, the merits and the possibilities of this method of analysis. Each of these methods of illustration has its advocates, the chief of whom to-day are perhaps the respective authorities here quoted—Arnold and Martens.



Bessemer Steel Ingot, showing blow-hole. Magnified 35 diameters.



Open-hearth Steel Ingot. Magnified 10 diameters.



Rolled Open-hearth Steel. Magnified 36 diameters.

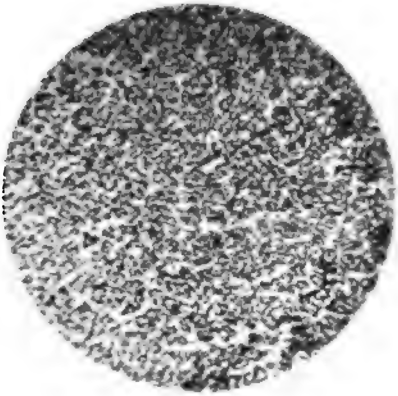
Spiegeleisen Crystals. Magnified 200 diameters.

Mild Steel after breaking in tension. Magnified 14 diameters.

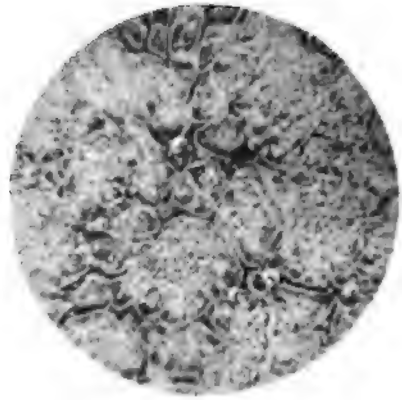
Mild Steel showing defects. Magnified 14 diameters.

PHOTOMICROGRAPHS OF STEEL. (After Martens.)

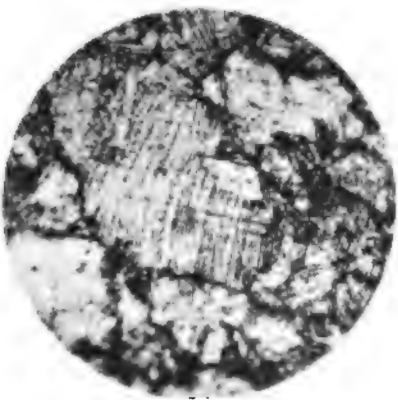
PLATE X.



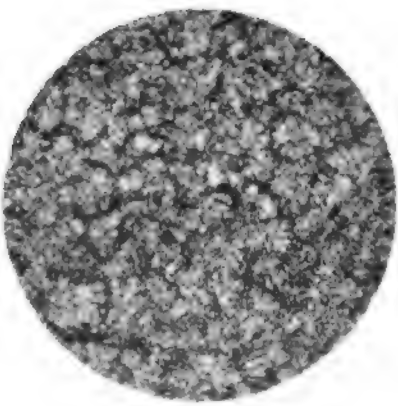
Thomas Steel Ingot.
Magnified 60 diameters



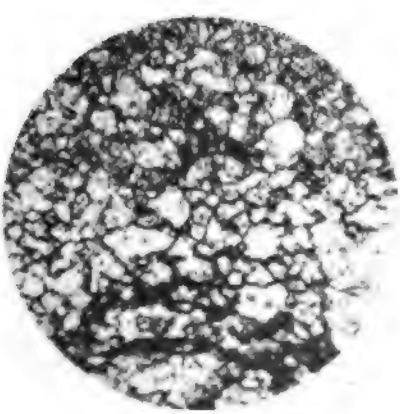
Rolled Manganese Steel, 10.6% Mn.
Magnified 80 diameters.



Open-hearth Steel Ingot.
Magnified 400 diameters



Rolled Manganese Steel, 10.6% Mn.
Magnified 8 diameters.

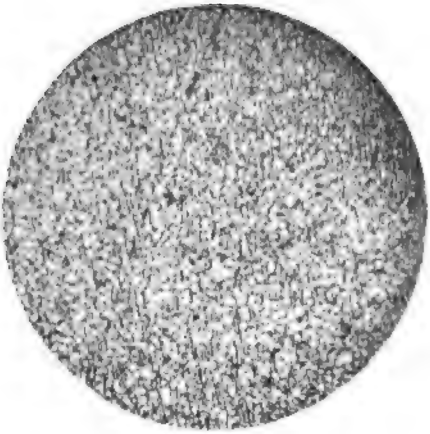


Open-hearth Steel Ingot.
Magnified 85 diameters.

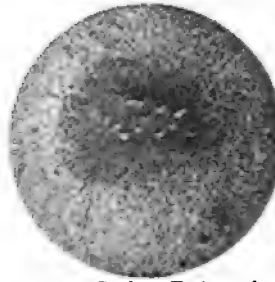
Cast Steel.
Magnified 85 diameters.

PHOTOMICROGRAPHS OF STEEL. (After Martens.)

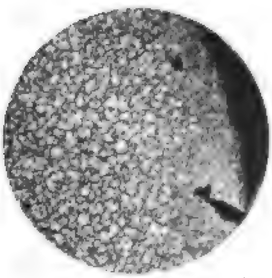
PLATE XI.



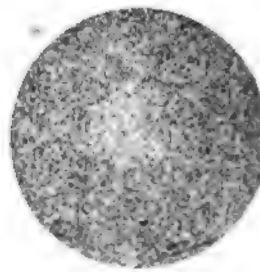
From a Steel-rail Web.
Magnified 10 diameters.



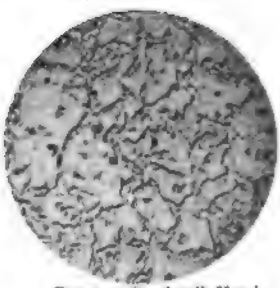
From a Broken Test-specimen, showing Defects. Mag. 14 diam.



From a Steel-rail Head.
Magnified 10 diameters.



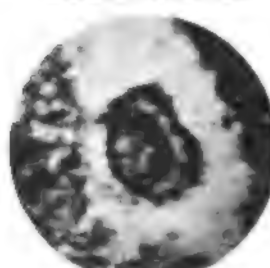
From a Steel-rail Web.
Magnified 10 diameters.



From a Steel-rail Head.
Magnified 70 diameters.

From a Steel-rail Head.
Magnified 200 diameters.

From a Steel-rail Web.
Magnified 200 diameters.



From a Steel-rail Head.
Magnified 1000 diameters.

PHOTOMICROGRAPHS OF STEEL. (After Martens.)

APPENDIX C.

COMPARATIVE ANALYSIS OF THE RESOLUTIONS OF THE CONVENTIONS OF MUNICH, DRESDEN, BERLIN, AND VIENNA, AND THE RECOMMENDATIONS OF THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS, WITH THE CONCLUSIONS ADOPTED BY THE FRENCH COMMISSION IN REFERENCE TO THE TESTING OF METALS.

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INTRODUCTION.

Two attempts have been made abroad to secure the adoption of uniform methods of testing construction materials—one under the auspices of technical conventions held at different times in Munich, Dresden, Berlin, and Vienna, where, besides the German members, there were present delegates representing various foreign countries; the other, in America, under the auspices of the American Society of Mechanical Engineers. Two reports presented to the Committee of Research, one by M. Polonceau and the other by M. Baclé, have given translations of the conclusions and recommendations thus adopted by the Conventions and by the American Society.

It is proper, therefore, to make a comparison between the resolutions adopted by foreign conventions and those adopted by our own commission, in order that we may examine the differences and analogies which they present. Such a comparison affords an opportunity of deciding whether there should be renewed study and research upon the points of difference, and of determining whether we may hope for international unity in the future, based upon the common resolutions already recommended and even now adopted in the current practical domestic relations of different countries.

The work of comparison in the present report, intended in some measure to form a starting-point for the studies of an international commission which might be created for this purpose, has been confined to the methods of testing metals, which is the sole subject of discussion in Section A. (Section des Métaux.)

To facilitate comparison, avoiding as much as possible all omissions, an attempt has been made to place together all resolutions relating to the same subject, and to this end the methods of classification used in our General Report have been adopted. The present report follows, therefore, the divisions and rules of the General Report, giving under each chapter only such resolutions as offer a possibility of comparison with the corresponding foreign recommendations.

The comparison is usually made by referring to the resolutions of the Conventions, for those are frequently reproduced in the recommendations of the American Society. In certain cases, however, those two groups of resolutions present differences which then become the subject of special mention.

It is advisable to point out in a general way, among the resolutions to be compared, a primary difference in principle (which is, however, of little importance) re-

lating to tests made upon finished pieces. The resolutions of the Conventions, identical in this respect with the resolutions of the American Society, define the tests which should be made upon certain articles in current use, such as tires, axles, and rails. For certain kinds of metals, such as wrought or cast iron, for different specified uses, they advise the discarding of such tests as to them seem unnecessary, while our Commission refuses to make any such elimination, feeling totally unauthorized to do so.

With regard especially to methods of testing, the studies of our Commission have been more general than those made abroad, and the recommendations that we have made include various kinds of tests not mentioned in the foreign resolutions, with which, therefore, no comparison is possible.

On the other hand, certain methods of testing have been especially recommended in all of the resolutions, such as tensile, shock, and bending tests, and it is with respect to those that the committee should make special comparative examinations.

In tensile tests the Conventions do not define so definitely as do we the quantities to be measured; they do not give the same limits of approximation in their measures; they follow the law of similarity in adopting cylindrical test-pieces, but use a different coefficient from ours, which tends to give to the useful or test length a greater value for the same diameter. Other differences are apparent in the description of their standard rectangular test-pieces.

The American Society prescribes an invariable standard of test length, independent of both the diameter and the cross-section of the test-piece.

The resolutions of the Conventions concerning shock tests are confined almost entirely to finished pieces. They give more explicit directions than do we in regard to the arrangement of the apparatus employed, but certain experiments made upon test-pieces that we have especially studied are not mentioned by them.

In bending tests they recommend that the bending should be done around a mandrel of unvarying diameter, while the American Society claims that the mandrel should vary in proportion to the thickness of the test-piece. The dimensions of their bars differ also from those used by us.

The differences just given are the most marked; but they are in reality of little importance, and probably can be easily overcome.

In the general resolutions concerning the precautions to be observed in preparing test-pieces we find at times differences of detail, but they are usually of minor importance, since the three sets of resolutions upon that subject are inspired by the same general principles.

In so far as the general formulas, as well as the methods of testing referred to above, are concerned, we find as a result of this comparison nothing to indicate that the desired unity of method may not be attained should an international commission be called for that purpose.

FIRST AND SECOND PARTS.

PHYSICAL EXAMINATION AND CHEMICAL TESTS.

The resolutions of the Conventions do not contain any specific directions upon those subjects, but they advise the acquisition of as thorough a knowledge as possible of the results of both microscopic and chemical examinations, especially in all cases of scientific research.

According to the recommendations of the American Society, the magnetic condition of a metal should be especially mentioned.

Those recommendations require, moreover, that when in tensile tests the ruptured section presents a cupel form the fact should be mentioned, the relative position of the edges of the cupels with reference to each of the pieces of the tested bar being indicated.

To determine the effects of tempering upon steel, the American Society recommends in addition that the bars should be heated to a bright red and immediately quenched in water at from 32° to 40° F. (0° to 5° C.)

Our Commission requires that the test-bars should be heated to a comparatively deep cherry-red and quenched in water at 28° C., the volume of water being great in proportion to the volume of the bars.

The resolutions of the Conventions also require that the metal should be heated to a cherry-red (placed by them at from 550° to 600° C.) and quenched in water at 25° C.

They recommend also, as will be seen later on, that copper test-pieces should be heated to 700° C. and quenched in water at 15° C.

That recommendation is not found among our conclusions, which require, however, that deductions from experiments with copper or its alloys shall be drawn from tests made after the last annealing in the manufacture of the plates.

THIRD PART.

MECHANICAL TESTS.—RECOMMENDATIONS COMMON TO ALL METHODS OF TESTING.

CHAPTER I.*—GENERAL OBSERVATIONS.

Our resolutions show the importance of accompanying mechanical tests of whatever nature with the tensile test; they indicate the approximate exactitude with which one should be content in the majority of current tests, by showing that in practical experiments the exaggerated precision required in scientific research is not necessary. They avoid, moreover, giving any indication regarding a choice between the various methods of testing according to the application proposed.

The resolutions of the Conventions recommend generally that testing should be done with reference to the work to which the pieces are to be subjected; at the same time they indicate which test to adopt and which to reject in specified cases; they advise the adoption of regular tests for certain pieces frequently employed, such as rails, tires, and axles for railroad use, cast or wrought iron, materials for shipbuilding, etc., recommending the greatest possible number of tests upon all pieces of the same delivery, testing without damaging them.

They recommend, for instance, testing tires and axles by shock with a standard impact machine, claiming that it is useless to test tires by a hand-hammer or axles by flexure. They add that it may be advisable to have recourse to tensile tests to obtain certain additional complemental information.

They confine themselves to recommending that the degree of exactitude attained in the tests should be stated, or at least that data should be furnished allowing it to be determined.

The resolutions of the American Society recommend that tests should be made as far as possible upon the pieces themselves whose quality it is desired to ascertain, being careful to reproduce as far as possible the conditions to which the pieces will be subjected when in use. They add, with a view to facilitate comparison, that it would be well to make a standard test over and above the tests upon finished pieces.

With regard to registering apparatus, our Commission limits itself to saying that it should be employed without hesitation, as its use even when it is not positively accurate may prevent serious error.

The American Society seems, however, to recommend it more formally, especially for tensile tests.

In regard to test-pieces that are imperfect, our resolutions require that they shall be thrown out of the calculation of averages established from a scientific point of view whenever there are exceptional or local defects in the test-pieces.

The American Society remarks on this subject that in making tests all pieces of abnormal appearance or those showing superficial defects should be rejected. If, however, a perfect piece cannot be found, a record of the imperfections should be kept.

CHAPTER II.—PREPARATION OF TEST-PIECES.

CONDITION OF THE METAL.

Our resolutions recommend in general that the condition of the metal employed shall be precisely defined principally with respect to hammer-hardening (*écrouissage*),†

* The "chapters" here referred to are the chapters of Part Three (Vol. I) containing the recommendations of the French Commission in reference to the Testing of Metals.

† Hammer or cold hardening (*écrouissage*) is an alteration produced by working a metal when cold with a hammer, a die, a punch, shears, etc. *Ecrouissage* is the standard change in a metal which has been subjected to a permanent deformation at a temperature lower than that required for annealing.

and especially when soft metals are used, that the test-pieces shall not be removed until the metal has been brought to the exact condition under which it is decided to test it; they mention also the precautions to be taken in finishing off test-pieces, in order to avoid any strain which might produce a change in the condition of the metal.

The American Society also insists upon the importance of observing its resolutions, which state that the lightest blow from a hammer, or any blow incorrectly given, may falsify results, should the abnormal strain thus produced be greater than that required to cause variation in the quality of the metal to be tested.

They admit, moreover, that soft metals are less susceptible to those influences than hard metals.

With regard to plates of mild steel, the Conventions have decided that it is unnecessary to test them after they have been annealed, as it is too difficult to determine the exact temperature of annealing. They add, in conclusion, and in accord with our Commission, that the object generally sought is to ascertain the nature of the metal in the condition when delivered.

THE PLACE AT WHICH AND THE MANNER IN WHICH TEST-PIECES SHOULD BE CUT.—FABRICATION OF TEST-BARS.

The resolutions of the Conventions require, for instance, with rails, that the bars detached shall have square* sections and contain the exterior fibres of the rail. Our advice is in general to take the test-bars from the thinnest and thickest parts of the final rolled section.

With regard to plates, the resolutions of the Conventions are imbued with the same spirit as ours, from which they vary only in minor details. They advise taking the test-pieces from the longitudinal and transverse sides, cutting away, when raw or uncut plates are used, at least 30 millimeters in width from the exterior; when dressed plates are used the test-pieces should be chosen from plane plates of regular thickness.

They admit, without going further into details, that the strips should be cut with shears or by a saw, but for bridge iron or boiler-plates they require that the strips cut off by shears shall be straightened out cold by a press, by the use of a wooden mallet, or by small hammers of lead or copper. Before cutting out the test-pieces the strips should be planed down on each side for a width of at least 5 millimeters to do away with the effect of the shears. Upon express demand annealing is permitted for straightening out strips cut by shears from boiler-plates.

Our resolutions impose more minute precautions to prevent the deformation of the bars, and if annealing is indispensable for straightening, they require that a temperature of 700° C. shall not be exceeded.

They require also that the test-pieces shall be cut from the strips by a machine, without, however, stipulating that a depth of 5 millimeters in thickness of the metal shall be removed.

For rolled products, the resolutions of the Conventions prescribe that the rough-rolled surfaces shall be preserved. Our instructions do not mention that point, but it may be observed that in practice that method is always followed.

The Conventions require, finally, that the reports of tests shall make known the source and the method of manufacture of the test-pieces, and they add that those pieces designed for compression tests should, as far as possible, be smoothed by planing or turning.

In the preparation of test-pieces of copper, they recommend the most minute precautions, which are also mentioned in our resolutions.

Besides those, they add, as has been indicated above, that bars of copper should be annealed at 700° C.† before they are completed, then cooled in the air to a dull red, and finally quenched in water at 15° C., while our resolutions demand that tests shall be made after the last annealing in the fabrication of the plates.

In regard to test-pieces of cast iron, the Conventions require that they shall be cast in a mould of very dry sand having an inclination of 10 centimeters per meter.

* The original resolutions require only a *rectangular* section.—O. M. C.
† That temperature must not be exceeded. See Original Resolutions.—O. M. C.

The entrance for the flow of the melted material should be placed 20 centimeters above the mould, determining thus the length of the runner-stick. Those test-pieces should be left with the rough surfaces produced by the moulds.

Our resolutions require that for pieces which at some places are more than 9 centimeters thick the test-bars should be cut by a machine from the foot of the runner-stick. For other pieces the test-bars may be cast separately, if care be taken to give the mould an inclination of about 20 centimeters per meter and to make the runner-stick from 15 to 20 centimeters long.

They always discard the runner-stick for bars cast in contiguous pieces.

The American Society remarks, without stating any general laws, that the superficial crust found upon raw materials, either rolled or cast, is either an advantage or a hindrance, according to the work in view, and should therefore be taken into account in the preparation of test-pieces.

Chapter III.—Study of the Influence of Temperature upon the Results of Tests.

Our resolutions point out the precautions to be observed in making tests at any given temperature, high or low; afterwards they deal especially with tests made at ordinary temperatures, remarking that the influence of variations in temperature is felt principally in shock tests, and giving directions for the mitigation of the same.

The resolutions of the Conventions do not contain any directions with regard to those tests; they require that in the cold bending of copper bars the temperature shall not be less than 10° C.

Chapter IV.—Study of the Influence of Duration.

Our Commission gives no positive rule with regard to the effect of the duration of test, considering that the subject if not as yet sufficiently understood; it recommends, however, the continuance of study upon that subject.

The Conventions declare for their part that the influence of time is incontestable, especially in tracing the diagrams in tensile tests, but they conclude that as yet they have not sufficient ground for establishing any fixed velocity of testing iron, copper, and bronze.

On the other hand, in cold-bending tests they claim that the duration is of no importance. In operating upon heated materials they require that the tests shall be made as rapidly as possible, but this is doubtless in consideration of the cooling of the bar.

The American Society requires that the duration of tests shall be noted.

Chapter V.—General Observations upon Testing Apparatus.

APPARATUS OPERATING BY GRADUAL ACTION.

Machines for Tensile Tests.

The Conventions confine themselves to demanding that machines properly handled shall not produce any shock on the test-pieces. They sanction the use of machines operated by hydraulic pressure or by a screw. They add that the test-pieces should be so mounted that the strain of tension or compression shall be uniformly distributed throughout the cross-section.

Besides those requirements, which coincide with those of our Commission, the Conventions include in their resolutions more explicit directions as to the mode of fastening the test-pieces for tensile tests than do we.

For cylindrical test-pieces they propose the use of spherical bearings, preferably in one piece.

They agree that test-pieces of rectangular cross-section shall be held at each extremity by a bolt passing through a slot provided for the purpose, or that the pieces shall be provided with milled heads and clamped by proper wedges.

They forbid positively the use of the serrated wedges used by us, and that point should be submitted to a renewed examination by us.

This last resolution is also adopted by the American Society, which, moreover, recommends in positive terms the use of two special types of attaching apparatus in use in the United States.

With regard to round test-pieces the American Society proposes to prolong the cylindrical section by conical bearings resting upon the clamping-pieces, in preference to threaded ends, such as are often used. This form is recommended for all metals except copper and its alloys, for which the Society considers it as yet impossible to give any standard type, conclusive experiments being lacking.

Machines for Pressure Tests.

The Conventions in their resolutions are in almost perfect accord with us on this subject. They require that the strain shall be carefully distributed throughout the cross-section, and point out that to attain such a result it is well so to place the pressure-plates that at least one may move easily and freely in all directions.

Those resolutions recommend, moreover, the use of very smooth test-pieces; in other words, those which have been planed or turned; but this recommendation has reference only to the bearing-surfaces, since in speaking of castings they require besides that the faces of test-pieces for flexure and compression shall be left in the rough state.

The American Society requires that the pieces to be tested shall be placed in position without the aid of any intervening medium whatsoever, such as wedges, supporting disks, etc., and that they shall be brought exactly into the axis of strain. That society repeats that the test-pieces should be prepared with the utmost care, the bearing-surfaces being exactly parallel and normal to the axis.

Whenever tests are made upon large pieces horizontally placed it is necessary to take into account the initial flexure due to the weight of the piece as held in place.

Machines for Transverse, Folding, Bending, and Curving Tests.

The Conventions confine themselves to recommending slow-moving apparatus, acting either by pressure on the middle between two supports or by lateral pressure brought to bear upon one part of the test-piece, while the other is securely held by clamping. Such apparatus should be simple and capable of being used rapidly. The weakest part of the test-piece should be clearly visible.

For folding, they simply indicate that it should be done in a continuous manner, and that if a mandrel is used it should be of the smallest possible diameter, recommending in certain cases one with a fixed diameter of 25 millimeters. They repeat, moreover, that the angle of bending is not alone sufficient to indicate the deformation, but that the radius of external curvature must be taken into account.

The American Society recommends the use of a very simple apparatus for making bending tests upon mandrels of varying diameter, and prescribes the use of the hand-vise for bending, which is accepted by us, however, with certain restrictions.

For transverse tests that society advises that supporting wedges shall not be used, preferring rolls that shall be displaced in proportion to the deformation by flexure of the bars.

The foreign resolutions do not point out the different modes of testing by bending that are specially defined by our Commission, but the Conventions state that the permanent committee should seek to determine, in the comparative tests remaining to be executed, the best method of measuring deformations.

Machines for Torsion Tests.

According to our conclusions, those machines should be arranged in such a manner that the axis of the piece will not sustain any flexure.

The resolutions of the Conventions give no data upon this subject, but the American Society has given definite instructions tending to prevent the production of any disturbing strains, such as transversal flexure or longitudinal tension over and above torsion properly so called. To this end the collars which hold the test-pieces should be exactly concentric with it, to avoid giving any but a tangential strain.

APPARATUS OPERATING BY ABRUPT ACTION.

The resolutions of the Conventions are generally in harmony with ours in principle, but they give much more explicit directions for the setting up of testing-hammers, especially in the case of heavy machines intended to test whole pieces. They have adopted for this purpose, as a standard type of hammer, one weighing 1000 kilograms, permitting in certain exceptional cases the use of one weighing only 500 kilograms, and they have decided that every hammer of the standard type shall be stamped and officially registered. They require that the studies relating to the question of shock tests shall continue, and they have charged the permanent committee with collecting all new propositions relating to the installation of machines for shock tests.

They accept hammers as we make them, of cast iron or forged steel; they add that the striking-surface should be of forged steel, finished by dovetailing and made secure by wedges in such a manner that the vertical centre of gravity of the whole may not be disturbed.

This vertical should coincide with the axis of the leads and should be indicated by marks upon the anvil or the anvil-block. The proper arrangement of the striking-surface should be verified by means of suitable reference-points.

We require, moreover, that the mass and shape of the hammer shall be perfectly symmetrical with respect to the plane of the leads.

Our directions, more especially with reference to tests upon test-pieces, regulate the form of the face of the small hammers used, and they indicate even the radius of curvature to be given that face, depending on the kind of metal to be tested. They also give the different weights for such hammers used under similar circumstances.

According to the resolutions of the Conventions, the guided length of the hammer should be at least double the clear width between the guides; we claim, however, only that it should be greater than the width between the guides.

The Conventions require that the weight of the anvil-block shall be at least ten times that of the hammer, and that the foundations shall be inelastic. We require that the anvil block shall constitute, either alone or embedded in solid masonry, a solid mass from fifteen to twenty times heavier than the hammer.

In the working of the detaching apparatus the Conventions agree with us that there should be no wedging. The Conventions advise the placing of the point of suspension upon the same vertical as the centre of gravity of the hammer, and to insert between the detaching device and the hammer a short flexible piece—for example, a chain or cord. They point out as a style to be recommended the detaching device adopted in Russia, which, however, from the sketch given, does not seem to be provided with an intermediate chain or cord

In regard to the friction on the leads, they recommend that those leads should be lubricated with plumbago. They reject all apparatus having a work due to friction greater than 2 per cent of the usual work. They describe a process of measuring friction by inserting a spring-balance between the hammer and its lifting-rope. They propose to deduce the effective weight of the hammer from the effect produced, with a given height of fall, on a centrally mounted standard cylinder made of copper, of dimensions yet to be determined.

For the height of fall they advise that 6 meters shall not be exceeded, as the setting up of hammers of greater height cannot be done with as much security or exactitude. They recommend the use of a sliding scale for measuring the effective work, so that the zero of the graduation may be set at the top of the piece to be tested. That scale should be divided into metric half-tons. (See Fig. 306, p. 378.)

All of the recommendations of the Conventions with regard to the setting up of machines for shock tests are agreed to by the American Society; the weight of the hammer and the height of fall are determined by the measures in use in America. The weights adopted are, respectively, 1000, 1500, and 2500 pounds for testing large pieces (equal to 453, 653, and 907 kilograms), and the height of fall is fixed at 20 feet (equal to 6.09 meters), allowing, however, without doubt, the adoption of a less height of fall in special cases. (See standard adopted by the National Car-builders' Assoc'n, p. 379.)

Chapter VI.—Examination of the Forces to be Measured.

Our resolutions recommend in effect that the strains and deformations produced in all tests by continuous action shall be measured; these form of necessity two great classes—the one of elastic and the other of permanent deformation.

Concerning the period of elastic deformation, three limits of elasticity are distinguished and defined, viz., the theoretical, the proportional, and the apparent. Concerning the period of permanent deformation, our resolutions define the maximum load supported, and the load of rupture, properly so called, giving the deformation corresponding to each of those two loads.

Neither the resolutions of the Conventions nor those of the American Society give any instructions common to all tests made by gradual action; they restrict themselves entirely to tensile tests.

The Conventions require that during the elastic period there shall be sought the yield-point and the limit of proportional elongation, appearing at the same time to admit that those two limits are blended, and during the period of permanent deformation the maximum resistance and the beginning of contraction as well as the load of actual rupture with the corresponding section.

The American Society requires the determination of the same information, excepting only the beginning of contraction, insisting especially upon the importance of the yield-point, which measurement it claims should be made with the greatest precision.

It defines that limit as being the load which produces a modification in the rate of elongation, which would seem to identify it with the proportional limit; but farther on, in an annexed illustration, it requires that the limit shall be determined by noting the point at which the elongation is suddenly augmented, which brings us back to the apparent limit.

The recommendations of that society prescribe, moreover, the measurement of the elastic elongation with a view of determining thereby the modulus or coefficient of elasticity, and they point out, to this end, a special process consisting of measuring the elongations between certain limiting loads, determined in advance.

They also observe the smallest section of the test-piece under the action of the elastic limit (yield-point), and, after the test, the section of rupture.

In the calculation of the strain brought to bear upon the unit of section, their resolutions add that it is necessary always to consider the initial appearance of the test-piece, and not the constantly-changing appearance under the different loads that it supports up to the limit of rupture.

This resolution conforms with that of our Commission. However, as in experiments made upon copper and brass, a comparison of the loads developed in the course of the tests with the corresponding deformations has given rise to interesting conclusions; our Commission has expressed the hope that analogous studies shall be made in regard to iron and steel.

Chapters VII and VIII.—Mechanical and Technical Terminology.

Those two questions have been examined by our Commission only, but we have not been able to establish any fixed laws in regard to them. They have not been considered by either the Conventions or by the American Society.

FOURTH PART.

DETAILED STUDY OF THE DIFFERENT METHODS OF TESTING.

FIRST CLASS.—METHODS OF TESTING BY GRADUAL ACTION.

Our Commission states in a general way that those tests should be made in a manner as continuous and as regularly progressive as possible, and that recommendation is in accord with that adopted by the Conventions with regard to tensile tests.

The American Society agrees to this also in a general way, but it provides for stopping the strain at certain intervals, whenever it is necessary to make observa-

tions of deformation, which is the case, for instance, in measuring the elastic limit and the elastic elongation. The Society requires in such cases that the developed strain shall never be diminished, but that it shall be maintained continuously in action.

CHAPTER I.—TENSILE TESTS.

MEASURE OF STRESS AND ELONGATION.

Our resolutions define the precision to be sought in the definition and scientific observation of the first two elastic limits previously described; they point out that for such purpose the elongations should be measured to the nearest thousandth of a millimeter.

Such precision is unnecessary for the determination of the third limit, called the apparent limit, or the beginning of great deformation under a constant load.

Regarding the observations made during the period of permanent deformation, our Commission recommends that the maximum load that can be borne shall be measured, saying that it does not appear to be necessary to measure the load of rupture, while, as has been shown heretofore, this measurement is required by the Conventions and by the American Society.

In regard to measuring elongation, our Commission believes that for current practical tests on products of the same well-known make it is sufficient generally to measure the total elongation after rupture; but in more exact tests it would be useful to make some special experiments with a view to determining the relative value of the different parts which compose the total elongation (distributed elongation and elongation of contraction).

The foreign resolutions do not consider this distinction. They recommend simply a method of measuring the total elongation suitable for reducing the results to uniformity by disregarding the influence that the position of the section of rupture beyond the middle third of the test-piece may have upon the observed elongation. This method amounts in principle to doubling the measured elongation for a distance equal to one half the length of the test-piece, counting from the section of rupture on the side, where it is possible to measure it, in such a manner as to render the same conditions as would have obtained had the rupture occurred in the middle of the test-piece and had the elongation been produced freely on both sides. The result is to increase a little the effective elongation measured directly on the real test-piece.

This method is inconvenient in that it is necessary in advance to divide the useful or test length into very small sections. The length of each section is fixed at 1 centimeter in Germany and at a half or even a quarter of an inch in America (12 or 6 millimeters). As the elongations during the course of the test are produced in a comparatively irregular manner upon the length of the test-piece, we may consider that they are not necessarily uniform at the ruptured section even when that is in the middle of the piece, and the method can give in this respect only approximate results.

It is true that if recourse to that method is not desired it will be found necessary, according to our resolutions, to discard every test-bar whose elongation of contraction is not integrally included between the reference-points; the resolutions of the Conventions, particularizing still further, give this same injunction when the section of rupture falls beyond the middle third of the gauged length.

In regard to tensile diagrams, our resolutions declare that it does not seem necessary to have recourse to them, at least in current practical tests with a view to ascertaining the quality of the metal tested from a determination of the useful surface they present.

The resolutions of the Conventions provide, however, for the use of diagrams, and require that their area shall be calculated up to the limit corresponding to rupture. They observe on this subject that without doubt, in principle, the work on the test-piece should be considered only up to the beginning of contraction, but that in most cases the work corresponding to that last period is of little importance, and the error thus produced cannot be very considerable. In cases where the diagram is not made by special apparatus, they advise making as many observations as possible during the test in order to trace the diagram by separate points.

The resolutions of the American Society also provide for the use of such diagrams.

With regard to the varying coefficients thus far proposed, our Commission finds it impossible to recommend them, and, moreover, the foreign resolutions make no mention of them.

In regard to the precision to be sought in measuring strains and elongations, our Commission declares that for strains less than 5000 kilograms a determination to within 25 kilograms is sufficient, going up to the limit of 50 kilograms, when the strain exceeds 5 tons; or, for the first class, to within one two-hundredth, and for the second to within less than one one-hundredth of the total sum.

The Conventions state that on their part they will accept an error of one tenth of a kilogram per square millimeter for tensile strains corresponding to the elastic limit (yield-point) and to that of rupture, which leads in practice, especially for the load of rupture, to a much smaller proportion of error than we have permitted.

The American Society makes no recommendation upon this subject.

In measuring the dimensions of test-pieces and elongations our Commission recommends a determination to within five one-hundredths of a millimeter in the case of dimensions equal to or less than 10 millimeters, and it accepts an approximation to within one tenth only when the length to be measured is greater than 10 millimeters; the limit is, therefore, to within more than five one-thousandths in the first case, and to within less than one one-hundredth in the second.

The resolutions of the Conventions recommend a degree of precision reaching one one-thousandth in measuring the elongation of rupture, and of one one-hundredth in measuring the contraction of area (considered, no doubt, as being the section of rupture itself).

They recognize, however, that in many cases these decimals are uncertain, and that it is not necessary to add others. They state that it is sufficient to take the dimensions of test-pieces to within one-tenth of a millimeter, which, considering the measurement of the thickness and the width of the section, gives a degree of precision inferior to that recommended by our Commission.

They require that the elongation shall be measured on two diametrically opposed sides of the test-bar, in order that the mean may be taken of the sum of the lengths obtained by measuring upon each part respectively the distance comprised between the corresponding reference-point and the section of rupture.

For rectangular test-pieces it is even proposed to make three measurements of elongation, taking them upon the two sides and upon one of the faces. The mean of the first two measures should be given and the last one should be stated separately.

The General Report of our Commission only mentions the use of elasticimeters, which are considered almost indispensable in determining the elastic limit (yield-point), but which seem less necessary for the simple measurement of the total elongation.

With the aim of measuring the section of rupture with all possible precision our resolutions propose that the measurement of the dimensions shall always be made at two opposite points, and that there shall be considered either the circle of mean diameter, or the rectangle of mean dimensions, according as the test-pieces have circular or rectangular sections.

DIMENSIONS OF TEST-PIECES.

In order that the proximity of the heads shall not interfere with the observed results, especially in the measurement of elongation, our resolutions require that the distance from the springing or end of the heads to the reference-points must be equal at least to the diameter or to the greater side of the transverse section of the test-piece; they consider that under such conditions the form of the heads is not important.

The Conventions limit themselves to remarking that for cylindrical test-pieces the actual length of the cylindrical part should exceed the test length by at least 10 millimeters,* which may be interpreted, doubtless, as imposing a uniform minimum

* The original resolutions require that the actual length shall exceed the test length by 20 millimeters.—O. M. C.

of 5 millimeters of waste length at each end, regardless of the diameter of the test-piece, which may be 10, 15, 20, or 25 millimeters.

The American Society gives regulations analogous to those of the French Commission. It requires that with round test-pieces the distances reserved beyond each reference-point shall be equal to or greater than a diameter. For flat or built-up square test-pieces its regulations require once and a half the width of the section or the side of the square.

In order to make a comparison of the total elongations, taken after the rupture of circular test-pieces of different design, our Commission has decided to establish a fixed relation between the transverse section and the useful or test length of the test-piece. This relation, deduced by the law of similarity, shows that the test length should be proportional to the square root of the cross-section, and that fundamental law has been admitted also by the Conventions. The only difference is in respect to the value of the coefficient adopted.

While our formula

$$l^2 = 66.67A,$$

$$\text{or} \qquad l = 8.18\sqrt{A},$$

resutls in giving a diameter of 27.64 millimeters to a test-piece, having a test length of 200 millimeters, the formula of the Conventions,

$$l = 11.3\sqrt{A},$$

leads to a smaller diameter for the same test length, for it gives in fact a diameter of 20 millimeters to a test-piece 200 millimeters long. In a general way this formula recommends itself on account of its great simplicity, inasmuch as the calculation of the linear dimensions is immediate; the useful or test length amounting always to ten times the diameter.

Our formula presents, on the other hand, the advantage of expressing the area by a simple number, for example, 600 square millimeters for a test-piece 200 millimeters in length, considering that it is better to seek simplicity in expressing the cross-section—rather than in indicating the diameter, because, no matter what number expresses the diameter, the difficulty of measuring it is always the same, whereas it is only the section which intervenes in the calculations, and the adoption of a simpler number to express the section greatly facilitates such calculations.

Notwithstanding the employment of the law of similarity, the two resolutions preserve well-defined normal types, to which they recommend that reference be made. These are four in number in the two cases.

Our standards have, respectively, test lengths of 70, 100, 141, and 200 millimeters; cross-sections of 75, 150, 300, and 600 square millimeters, and diameters of 9.77, 13.82, 19.55, and 27.64 millimeters. The standards of the Conventions give sections very closely resembling these, but their longitudinal dimensions are greater; they have lengths, respectively, of 100, 150, 200, and 250 millimeters for diameters of 10, 15, 20, and 25 millimeters.

Those standard types are adopted for steel and iron, as well as for copper and its alloys; for castings, however, the Conventions prescribe test-pieces having a diameter of 25 millimeters, and a useful or test length of 200 millimeters.

The American Society prescribes four standards with diameters, respectively, of 0.4, 0.6, 0.8, and 1 inch, which equals 0.010, 0.015, 0.020, and 0.025 millimeter, but it preserves a constant useful or test length of 8 inches (0.2) meter, ignoring the differences which must occur in the measurement of total elongation by testing with a uniform length and a variable diameter.

The test length of 8 inches, as well as the various diameters given, have been chosen, moreover, for the purpose of approaching as nearly as possible the measures of the metric system, counting 8 inches as equal to 200 millimeters. This is expressly stated by the Society itself.

The law of similarity thus admitted, as has been shown, for cylindrical test-pieces, is extended by our Commission, with the same coefficient, to test-pieces of

square cross-section, and even to pieces that are simply rectangular, observing carefully certain restrictions established with a view to facilitate manufacture, by establishing different series, in each one of which the width remains the same.

The Conventions also extend the formula expressing the law of similarity to test-pieces of simply rectangular cross-section, but without introducing any definite restrictions. They recommend, however, that the section 30 by 10 shall be considered as the standard, even when the breadth and thickness may be chosen at pleasure.

When the thickness is given, as for plates, and when it does not exceed 24 millimeters, a width of 30 millimeters should be adopted.

Whenever the thickness exceeds 24 millimeters it should be considered as breadth, and a thickness of 10 millimeters should be given to the test-piece.

When the testing-machines are not sufficiently powerful, the limit of 24 millimeters should be replaced by that of 16 or 17 millimeters.

The American Society permits on its part a width of 1.2 inches (0.030 meter) for rectangular test-pieces having a thickness of less than 1 inch (0.025 meter).

If the thickness reaches 1 inch, the measure of 0.8 inch (0.02 meter) will be taken for the width.

In testing sheets and plates the crust produced in rolling should not be removed.

In testing sheets there should be taken two samples having the total section of the bar.

Besides the preceding rules just given, which are considered applicable only to rectangular test-pieces having a thickness of more than 5 millimeters, our Commission has adopted special dimensions for test-pieces having a thickness of less than that figure, considering that it is not necessary to be guided by the law of similarity, because that law is doubtless no longer applicable in such a case.

The useful or test length of those thin test-pieces has been fixed uniformly at 100 millimeters, without reference to the thickness or the nature of the metal to be tested.

CONDUCT OF TESTS.

Our resolutions recommend, as has been indicated, operating by progressive or gradual tension. This recommendation is also found in the resolutions of the Conventions, but the American Society permits interruption of the test, without relieving, however, the action of the strain, for the purpose of making certain important observations during the course of the test, such as those relating to the elastic limit (yield-point).

In general, the American Society recommends the utmost care in placing the test-pieces, so that they may be directly in the axis of the machine, and to this end it advises that it should be referred carefully to two normal planes intersecting each other in line with this axis.

The Society also recommends placing test-pieces for tensile tests under a very slight initial strain (from 0.7 to 1.4 kilograms per square millimeter) before commencing observations, in order that the errors generally made at the beginning of a test may be avoided.

In regard to the rate of testing, our Commission has confined itself to giving some approximate directions, and it observes, to that end, that the duration of a test, which should be in a certain measure a function of the volume of the test-piece, should be comprised between one and six minutes for current tests on test-pieces of ordinary dimensions. This time may be reduced to thirty seconds for small test-pieces having a thickness of less than 5 millimeters. Care should be taken, especially with soft metals, to avoid heating the bars.

The foreign resolutions make no recommendations upon this subject. The Conventions, however, observe that in establishing the diagram of tensile test great importance should be attached to showing with what rapidity the diagram was traced.

Chapter II.—Pressure Tests.

TESTS ON SHORT PIECES.

For determining the elastic limit our Commission advises the employment of cylindrical test-pieces having a diameter of 27.65 millimeters (600 square millimeters in cross-section) and 100 millimeters of useful or test length; but for the determination of the maximum resistance to compression or crushing the diameter of the standard test-pieces will be reduced to 19.56 millimeters (300 square millimeters of cross-section), and the useful length to 20 millimeters.

The regulations of the Conventions propose in testing castings the use of cubes of 25 millimeters a side, making them serve as samples for pressure-tests. In another passage, however, they recommend a height of 30 millimeters.

They give no complementary information on the subject of that test, merely stating that Wachler adopted 25 millimeters as the standard dimension in his studies.

The American Society considers that pressure tests upon short test-pieces are of little interest, recommending, preferably, the adoption of pieces of a length of from 10 to 20 diameters.

However, when it is a question of determining the resistance to disaggregation, the use of cylinders 1 inch in diameter (0.025 meter) and 2 inches in height (0.050 meter) is proposed. Whenever the elastic resistance is to be determined the height will be increased to 10 or 20 inches (0.254 meter or 0.508 meter), always keeping the useful or test length at 8 inches, as in tensile tests.

TESTS ON LONG PIECES (FLAMBEMENT).

Our Commission insists especially upon the interest of making buckling tests, stating that for iron and steel the resistance determined by the tests thus made is in no way proportional to that determined by tensile tests, and in the existing state of science cannot be deduced from the results obtained by the usual tests.

The Commission declares that the tests can be made on riveted pieces or on bars cut up for this purpose, and it observes that to render the two tests comparable it is sufficient that the ratio of the length of the test-piece to the minimum radius of gyration of the section should have the same value. In tests upon test-pieces the Commission proposes to give to that ratio values which are multiples of 5 or 10.

It recommends that those tests shall always be made under well-defined conditions, such as those by perfect hinging or complete clamping. The precautions to be observed in the first case are prescribed, but it is added that no satisfactory disposition for such testing by clamping is yet known.

The foreign resolutions give no particular instructions regarding pressure tests on long pieces. However, the American Society provides for tests upon pieces having a diameter of 1 inch and from 10 to 20 inches long for determining the elastic limit (yield-point).

It requires that the process shall be the same as in tensile tests, dividing the useful or test length into small sections in such a manner that the loss in value sustained may be determined from its elements, and that the calculation of the modulus and the coefficient of elasticity shall be made under like conditions.

Chapter III.—Transverse Tests.

TESTS ON TEST-PIECES.

astings.

Our Commission adopts for standard bars a section having a side of 40 millimeters, indicating that the total length should be determined in such a manner as to give a useful or test length of 150 or 500 millimeters according to the apparatus used (Monge or Joëssel balance)

The Conventions have adopted a prism 3 centimeters on a side, with a total length of 110 centimeters, giving a useful or test length of 100 centimeters. They

recommend measuring the resistance to flexure up to the point of rupture and the corresponding work on three test-pieces. The faces of the pieces should be left in their rough condition.

Our Commission prescribes that the faces of the bars shall be shaped by a machine and the edges rounded by a file. It regulates finally the duration of the test, which should be comprised between one and two minutes.

The American Society requires for scientific tests that the effect of superficial quenching shall be avoided by using bars measuring at least 2 inches on a side (0.050 meter) or 2½ inches in diameter (0.063 meter). In current practical tests there will be used bars measuring only 1 inch (0.025 meter) on a side, taking the necessary precaution to throw aside surfaces which have been hardened or marked with blow-holes.

Steel Plates for Springs.

Our Commission fixes the length to be adopted for developed test bars at 1 meter, the section to be used, however, being preserved without any modification.

The plate will be prepared under the same conditions as to quenching and annealing as the springs.

For testing, it will be placed on two movable slides, the use of which is also recommended by the American Society.

The test will be continued without stopping or lessening the load and with a regular and continuous movement up to the limit fixed upon, in such manner that the duration of the whole test is comprised between two and five minutes.

The foreign recommendations contain no special remarks upon that subject.

The American Society recommends, however, in a general way, regarding transverse tests, that the sample should be placed firmly in a horizontal position to avoid supporting wedges, that the strain shall be brought to bear exactly in the middle and normal to the axis of the piece, and in a plane passing through the three strained points. It requires that the bars used shall be 1 inch (0.025 meter) on a side, have a length of 40 inches (1.016 meters), and that they shall be placed upon supports 36 inches apart (0.914 meter).

According to the resolutions of the Society, the deviations should be measured from a fixed and invariable base. It is well to determine them at points situated at stated intervals from the middle of the bar in such a manner as to obtain certain elements of the curve of flexure over the whole extent of the length, and especially in the vicinity of the elastic limit (yield-point).

Certain values of the permanent deviation may also be determined.

TESTS ON FINISHED OR WHOLE PIECES.

Springs of Parallel and Spiral Plates.

Our Commission recommends in transverse tests that springs be subjected to a less strain than that which corresponds to permanent deformation.

The American Society, on its part, proposes to apply the maximum working load and to determine the deviation produced by that test. The test load will be obtained by proceeding by pressure or by shock, according to circumstances, but it will be applied only once.

Rails and Fish-plates.

Our Commission recommends determining as nearly as possible the load corresponding to the proportional limit of elasticity in testing those pieces; it adds that the duration of the tests should be comprised between two and five minutes.

The foreign resolutions examine also the question of transverse tests by static pressure, requiring that they shall be considered from the two following points of view: First, there will be determined the elastic limit (the point where the set becomes permanent); second, there will be measured the flexibility under increasing loads, exceeding even the elastic limit.

The American Society states that it is hardly practicable to subject either rails or axles to a transverse test by static pressure.

Chapter IV.—Folding, Bending, and Curving Tests.

Our Commission proposes to adopt for those tests standard bars 40 millimeters wide and 250 millimeters long, observing that the length may be reduced to 150 millimeters for copper and its alloys. It recommends, besides, that those various tests shall be made by a machine so arranged as to produce a slow progressive strain, and that in folding tests the initial fold shall be obtained with a radius of curvature differing as little as possible from that exacted for the limiting fold.

For mechanical folding the use of the hydraulic press or hammer is recommended.

Our Commission gives, besides, certain directions for folding done by a hand-vise. It is necessary to observe here that this mode of testing is prohibited by the American Society.

In regard to bending tests, our Commission has not yet fixed upon any stated diameter for the mandrel.

For curving tests it proposes to adopt such machines as will permit the measurement at each instant of the developed strain and the corresponding deformation.

The Conventions recommend, as has been indicated, the use of slow-moving apparatus, acting either upon the middle or at the extremity of the test-bar, but they require especially bending around a mandrel for the hot and cold folding test of irons and steels of all kinds, fixing upon a diameter of 25 millimeters for mandrels to be adopted in the case of wrought or cast-iron for bridges and of boiler-plates more than 6 millimeters in thickness. The mandrel should not be removed during the test until the ends of the test-pieces have become parallel to each other, and the bending will be continued to the point of contact in the case of test-pieces of copper.

They add that the test-pieces employed should have a rectangular section, the width of which should be three times the thickness, and that the edges should be slightly rounded.

They observe, besides, that the angle of folding alone is not sufficient for judging of the degree of deformation of the test-piece, but that it is also necessary to take into account the radius of external curvature. They propose to make this measurement direct by means of templets, or to deduce it from the measure of the elongation upon the exterior face.

They require, finally, that the permanent committee shall search for the most convenient method of measuring deformations, and they also require that it shall look into folding tests made upon defective pieces.

The American Society also recommends the bending test around a mandrel, but it prescribes the use of mandrels of variable diameters, and it recommends giving them a diameter equal to twice the thickness of the bar to be tested.

It approves, finally, the use of a very simple apparatus which shall permit that test to be made rapidly. It fixes 1 inch (0.025 meter) as the standard width to be given test-pieces for folding tests.

Chapter V.—Torsion Tests.

Our Commission confines itself to expressing the hope that experiments shall be continued with regard to the study of torsion.

The American Society has given certain instructions, previously indicated, with regard to the installation of apparatus to be used in those tests.

With regard to test-pieces, it recommends the use of cylindrical heads, prohibiting absolutely square-shaped heads. It gives a sketch of the position to be given the fastenings, and it demands that the distance between the head and the nearest point of reference shall not be less than one diameter.

It gives nothing concerning the determination of the useful or test length.

Chapter VI.—Mixed Tests (Shearing and Punching).

Our Commission has expressed the wish that the study of those tests may be continued, and it has proposed certain recommendations which may serve as a beginning for subsequent regulation.

The resolutions of the Conventions say nothing upon this subject; they recom-

mend only that certain plates, like those of wrought iron for boilers, shall be submitted to the punching test, but they claim that the test is useless for plates of mild steel or ingot iron, which should never be punched.

The American Society remarks that in this test special care should be taken to determine the space to be reserved between the edge of the finished bar and the edge of the hole punched in the interior of the section, in order that cracks may not be formed.

This question is being studied by numerous experts both in France and in America.

SECOND CLASS.—METHODS OF TESTING BY ABRUPT ACTION.

Our Commission, in common with the foreign resolutions, indicates the particular interest to be found in shock tests, for they give information which gradual action cannot furnish. It recommends that study concerning tensile tests by shock and the use of explosives may be continued.

CHAPTER I.—TRANSVERSE TESTS BY SHOCK.

Our conclusions relate especially to tests on test-pieces, as has been shown above. They fix the dimensions to be given to test the bars, according to the nature of the metal tested, regulating under the same conditions the weight of the hammer, the form of the striking face, and that of the edges of the supports.

They recommend, besides, a continuance of theoretical research, to ascertain the respective effects of the two elements which are the component parts of the energy of shock, namely, the weight of the hammer and the height of fall. They require that for each metal there shall be determined the height of fall above which the fragility increases rapidly, that there shall be studied the effect of constantly increasing heights of fall in ordinary transverse tests made on test-pieces resting upon two supports, and finally that there shall be studied tests made by bringing the shock to bear upon the free end of test-pieces clamped at one end only.

Those different subjects are not examined in the foreign resolutions, which give special attention to tests upon whole pieces, and determined, as has been indicated, the weight of the hammer and the height of fall to be used.

In tests upon finished pieces the resolutions of the Conventions recommend the use of bearing blocks or caps of such form that the upper surface of the piece shall be perfectly horizontal, the face of the hammer being always perfectly smooth and plane. Those pieces should be as light as possible.

Our Commission appears to think, however, that the use of those caps may be dispensed with when it is a question of testing pieces before sustaining in service blows which would be of such a character as to alter them.

In a paper on shock tests, presented to the American Association for the Advancement of Science at its meeting of August, 1894, in Brooklyn, by Prof. Mansfield Merriman, vice-president of the society, the author recommends the disuse of those caps; he proposes to give the hammer a large striking surface to avoid loss of energy by heat, and to take account of the rebound of the hammer.

The Conventions claim, on the other hand, that the data derived from tests thus far made are not sufficiently conclusive to admit of giving any fixed form for either the supports or the pieces destined to receive the shock.

In observations on the results they declare that it is sufficient to determine the curvature deflection to within about 1 millimeter, when it is measured on a cord of from 1 to 1.5 meters in length.

In a general way they recommend, as has our Commission, the taking of careful notes regarding all the peculiarities of the test, stating, for example, whether there had been any interruption during the test, whether the piece had been turned over, etc.

CHAPTER II.—SUPERFICIAL PENETRATION TESTS BY SHOCK.

That mode of testing, minutely studied by our Commission, has not been examined in the foreign resolutions.

Chapter III.—Perforation Tests by Shock.

Our Commission has only been able to express a desire that new experiments should be made for the purpose of solving the various questions which have been brought up by that method of test. The study of that method of test has not, as yet, been broached by the Conventions or by the American Society.

THIRD CLASS.—STUDY OF HARDNESS AND FRAGILITY.

Chapter I.—Proposed Definitions and Methods of Measuring.

Our Commission has reserved the study of that subject for a future session.

Chapter II.—Tests of Hardness by Scratching and by Resistance to Wear and Tear.

Our Commission has expressed the hope that those two subjects might be the object of complemental studies.

The Conventions have expressed a like hope in reference to the wear and tear of rails and tires, requiring that those tests shall be made under conditions as nearly like those of practice as possible.

The American Society declares that it is impossible to present any recommendations upon that subject; however, it points out that for rails it is proper to make tests upon curved pieces, and to take into consideration the action of shock, of rapid rolling, and of variations in temperature and humidity.

Chapter III.—Folding Tests after Cold Hardening by Punching, or after Cutting.

The Conventions have charged the permanent committee with searching for the causes of irregularities in ingot iron shown by unexpected breakages, notwithstanding tests upon the broken pieces had given satisfactory results.

Our Commission considers that folding tests made after cold hardening (*écrouissage*) or cutting should show those irregularities, and it has given in this respect certain practical rules which may in some measure bring out the information desired by the Conventions.

FOURTH CLASS.—TESTS OF MANUFACTURE.

Chapter I.—Cold-working Tests.

The enlarging upon the mandrel, the heating, the flattening, and crushing tests, all studied in our resolutions, are not mentioned by the foreign resolutions.

Chapter II.—Hot-working Tests of Irons and Steels.

TESTS BY BENDING AND FOLDING.

Our Commission points out certain tests to be made on plates, angle-iron, and facing-iron; for bars cut from plates the same dimensions are assumed as for cold folding.

It points out that all these tests should be made at the same heat.

The Conventions recommend that certain kinds of metals, such as ingot or wrought iron for bridges, and plates of ingot iron or mild steel for boilers, shall be subjected to the hot-folding test. For the latter a test will also be made after quenching The hot test is made around a mandrel under the same conditions as in cold bending.

TESTS BY STAMPING, BENDING INTO HOOKS, BORING, PRESSING, FORGING, FLATTENING, AND WELDING.

The greater part of those tests are not mentioned in the foreign resolutions, except those by flattening and welding.

The Conventions recommend those two modes of test for different kinds of metals without trying to establish any definite rules.

They declare that it is difficult to generalize from tests by welding, since much depends upon the skilfulness of the operator, and they have determined to require a subcommittee to study the utility of the various hot tests.

The American Society requires recourse to be had to the flattening test for testing riveted bars, and it states, as does our Commission, that the amount of spreading out obtained before the appearance of fissures furnishes one measure of the quality of the metal.

It insists strongly on the welding test, stating that such a test has a special importance in the United States, inasmuch as welded pieces are frequently used there.

It gives the precautions to be observed in welding, which should be done at one temperature, a white heat, and it requires that after the operation the test-piece shall be submitted to a tensile test.

Another sample should be submitted to a folding test after a groove has been made of a depth equal to that of the weld.

The American Society demands that the welded bars shall be allowed to cool without being wet.

It does not prescribe the boring or piercing test after welding recommended by our Commission.

FIFTH CLASS.—SPECIAL TESTS ON CERTAIN FINISHED PIECES.

The foreign resolutions insist strongly upon the great interest that there will be in being able to make tests upon the finished pieces themselves; they require that in setting up testing-machines the possibility of making tests on finished pieces shall be kept in mind. They add that the shock test is generally the most important, and for certain pieces in frequent use on railroads they recommend or prescribe certain special tests.

Our Commission, however, does not feel at liberty to formulate any such resolutions, and in making its studies of tests on finished pieces it confines itself to pointing out the method of executing the tests without indicating that any one may be superior to the others.

CHAPTER I.—TESTS ON WIRE.

Our Commission has studied the principal tests that may be made on wire, i.e., tensile, folding, winding, and torsion.

The Conventions require upon this subject only that the torsion test shall be made by means of suitable machines, and that the folding test shall be made by machinery bending the piece alternately in two opposite directions around a mandrel 5 millimeters in diameter.

The American Society reproduces those conclusions, requiring, however, that the diameter of the mandrel shall be equal to that of the wire.

The analogous test studied by our Commission is that of winding or wrapping. It requires that the diameter of the roller or spool shall vary according to the destined use of the thread, but requires that it shall be always a multiple of the diameter of the wire.

CHAPTER II.—TESTS ON WIRE ROPE.

Our Commission prescribes the rules to be followed in tests for tension and flexibility. The Conventions and the American Society require that a tensile and a shock test shall be made in the longitudinal direction without giving any further details on that subject. Our Commission expresses the hope that the study of ten-

sile tests by shock may be continued, as well as the folding or winding tests, in order to furnish information as to flexibility.

The Conventions add that the folding test is of value only when continued for a given length of time.

Chapter III.—Tests on Chains.

Our Commission proposes submitting chains to a tensile test made at first under a moderate load, then carried to rupture, and it gives some instructions relating to the execution of that test. For the test under a moderate strain it proposes adopting a strain double that to be met with in actual service.

The American Society requires only that the chain shall be subjected to the service strain, and that the elastic and permanent elongation shall be measured as well as the change in the form of the links.

Chapter IV.—Tests on Rivets.

As a special test on bars for rivets, the American Society presents only the hot flattening test.

Our Commission gives two principal tests: one to separate the two riveted bands or bars by means of a chisel with a given bevel driven by blows of a hammer, and the other to subject those bars to a sort of shearing test.

Chapter V.—Tests on Pipes and Tubes.

The foreign resolutions give no special information in regard to the manufacture test to be made on pipes or tubes.

Chapter VI.—Tests by Hydraulic Pressure.

For tests of steam-boilers the American Society recommends the adoption of the method used by the Hartford Inspection Company, which is already in general use in the United States.

Our Commission, without prescribing the methods of test required in France, points out the precautions to be observed and the verifications to be made in testing boilers.

For testing cylinders and pipes the American Society proposes the application of a pressure equal to the maximum working load. It requires, besides, that the dilation of pipes under that load shall be observed, as well as the permanent dilation, if any be produced, and to mention whether the pipe leaks.

In conclusion, our Commission recalls the fact that hydraulic pressure has been used recently to acquire desired data relating to the elastic deformation of metals; and it expresses the hope that those studies may be extended to plates.

APPENDIX D.

SPECIFICATIONS FOR STRUCTURAL STEEL.

I. PROPOSED BY A COMMITTEE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, 1896.

THE following specifications for structural rolled and for cast steel are those recommended by the Committee of the American Society of Civil Engineers (1896):

Tensile strength	Low steel......	60,000 lbs.	± 4,000
	Medium steel..	65,000 "	± 4,000
	High steel.....	70,000 "	± 4,000

Yield-point = 55% of the ultimate resistance of specimen.

$$\text{Per cent elongation in 8 in.} = \frac{1,500,000}{\text{Ultimate}}.$$

$$\text{Per cent reduction of area} = \frac{2,800,000}{\text{Ultimate}}.$$

Rivet-steel, when heated to a low cherry-red and quenched in water at 82° Fahr., must bend to close contact without sign of fracture. Specimens of low steel when treated and tested in the same manner must stand bending 180° to a curve whose inner radius is equal to the thickness of the specimen without sign of fracture. Specimens of medium steel, as cut from bars or plates and without quenching, must stand bending 180° to an inner radius of one and one half times the thickness of the specimen without sign of fracture; while those of high steel, also without quenching, must stand bending 180° to a radius of twice the thickness of the specimen without sign of fracture.

STEEL CASTINGS.

In steel castings the tension test is recommended, with the following requirements:

Ultimate.................	65,000 lbs. per square inch.
Yield-point..............	35,000 " " " "
Elongation in 8 in........	= 15%
Contraction..............	= 25%

The criterion for the elongation is that of Tetmajer, which consists in calling the product of the ultimate strength by the percentage of elongation the "coefficient of quality." The locus of this equation for the elongation, as given above, is shown in Fig. 74, p. 159, together with the ordinary limits of the elongation and a modified equation proposed by the author.

II. SPECIFICATIONS PROPOSED BY H. H. CAMPBELL.

(In his work "The Manufacture of Structural Steel," 1896.)

General Provisions on Methods of Testing.—(1) Rivet-rods and other rounds are to be tested in the form in which they leave the rolls, without machining.

Note.—This is apparently opposed to the recommendations of the International

Conferences,* wherein it is proposed that round test-pieces shall be turned to one of four standard diameters, with shoulders and screw-grip thread at each end, but it is stated elsewhere in the reports of the committee that only pieces for scientific investigation are to be prepared in this manner.

(2) Test-pieces from angles, plates, shapes, etc., shall be rectangular in shape, with a cross-sectional area of about one-half square inch, and shall be taken so that only two sides are machine-finished, the other two having the surface which was in contact with the rolls in the last pass.

Note.—The report of the committee above mentioned recommends this method of cutting tests, but specifies that there shall be shoulders at each end. This necessitates considerable extra machine-work without any notable effect upon the result. The limitation of the area of the piece prevents the passing of inferior material by an unusual increase in width.

(3) Should fracture occur outside of the middle third of the gauge length, the test is to be discarded as worthless if it falls below the standard.

Note.—This provision is copied from the report of the above committee, and is much to be commended. A deficient elongation when the piece breaks near the end is not the fault of the material, but a mere accident. On eye-bars, a failure in the eye should condemn the method of forming the head rather than the quality of the steel.

(4) In case one test-piece falls slightly below the requirements in any particular, the inspector shall allow the retesting of the lot or heat by taking four additional tests from the same lot or heat, and if the average of the five shall show that the steel is within the requirements, the metal shall be accepted.

(5) Drillings for chemical analysis may be taken either from the preliminary test-piece or from the finished material; but if the sample be taken from the centre of a sheared or universal-mill plate, the maximum limit of both phosphorus and sulphur shall be raised 25 per cent; e.g., from .04 to .05 per cent, or .08 to .10 per cent.

(6) The pulling speed of the machine for breaking test-pieces shall not be less than one-quarter inch per minute nor more than three inches per minute.

(7) The elastic limit shall be determined by the dropping of the beam.

Classes of Steel Proposed.—The following specifications do not deal with metal for special purposes, like gun-carriages, armor-plate, etc., but are intended to cover more or less fully the needs of the structural engineer. I do not expect that they will ever be adopted in their entirety as standard requirements, but this seems to be the simplest form in which to condense the investigations that have been recorded in the foregoing chapters, and to present the variations in the physical properties caused by changes in the history and section of the test-piece.

Engineers who do not wish to cumber their specifications with so many allowances for thickness and section will find herein the reason for many troublesome questions arising in the testing of the material, for I have tried to tabulate, as fairly as can be estimated, the effect of conditions that are ruled more by the laws of nature than by the skill of the manufacturer.

At the same time it will be found that the matter is not as complicated as would be indicated at first sight, for one page of general provisions and one page of physical limits for each kind of steel can hardly be called a very voluminous document to cover the specifications upon structural shapes.

The engineer who will compare the proposed requirements with what is demanded in other countries will find a remarkable difference. The specifications which are in general use in Germany are as follows†:

For Rivets.—Ultimate strength from 51,200 to 59,700 pounds per square inch; elongation 22 per cent in eight inches.

For Other Structural Material.—Lengthwise tests: Ultimate strength from 52,600 to 62,600 pounds per square inch; elongation 20 per cent in eight inches.

Crosswise tests: Ultimate strength from 51,200 to 64,000 pounds per square inch; elongation 17 per cent in eight inches.

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* *Report of Committee on Standard Tests to the Am. Soc. Mech. Eng.*, Appendix V.

† *Normalbedingungen für die Lieferung von Eisenconstruktionen für Brücken- und Hochbau* (Otto Meissner, 1893); also, *Ueber die Arbeiten der Flusseisen-Commission* (F. Kintzlé, 1892).

These are given as the limits accepted by the leading engineering societies of that country, and I am informed by Mr. Kintzlé* that they represent the general requirements at the present time for all classes of material. It is safe to say that if American engineers were satisfied with the German standards, there would not be one rejection for deficient ductility where there are twenty under our more rigid requirements; and if they would be content with a steel having an ultimate strength between 52,000 and 62,000 pounds per square inch, there would not be one fifth the number of heats discarded for being outside of the tensile limits. The bearing of these facts upon the cost of the material is self-evident.

I do not advocate any sacrifice of strength to economy, but I would impress upon American engineers that this soft metal is eminently suitable for structural work, while by maintaining their present chemical limitations and their requirements concerning ductility they will be assured of a material which is equal in quality to any produced in the world.

CLASS I.—EXTRA DEAD SOFT; FOR COMMON RIVETS, WIRE CABLES, AND OTHER PURPOSES WHERE EXCEPTIONAL TOUGHNESS IS REQUIRED.

Method of manufacture: Basic open-hearth process.

Chemical composition, in per cent: P below .04; S below .06; Si below .01; Mn below .50

Physical requirements as follows:

Shape.	Diameter in Inches.	Ultimate Strength, Pounds per Square Inch.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.
		Minimum.	Maximum.			
Rivet-rods	5/8	46000	55000	64.0	28.0	52
"	3/4	46000	54000	63.0	29.0	58
"	7/8	45000	54000	61.5	29.25	56
"	1	45000	54000	60.0	29.50	54
"	1 1/8	44000	54000	58.5	29.75	52
"	1 1/4	44000	54000	57.0	30.00	50

A rolled round about three-quarters inch in diameter, after being nicked about one-quarter way through, shall bend completely double without fracture, with the nick on the outer curve of the bend.

Heats rolled into bars less than five-eighths inch in diameter may be tested in trial rods of three-quarters inch.

If any bar fails to pass the physical tests, four more pieces shall be taken from the same heat, and the average of all five bars shall be considered the true record.

Rivets, when cut out of the work into which they have been put, shall show a tough silky structure, with no crystalline appearance.

See also general provisions, p. 756.

CLASS II.—BRIDGE RIVETS; FOR RIVETS IN RAILROAD BRIDGES.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .04 in acid steel, below .03 in basic; S below .05; Si below .01; Mn below .50.

Physical requirements as follows:

Shape.	Diameter in Inches.	Ultimate Strength, Lbs. per Square Inch.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.		Average Reduction of Area, Per Cent.
		Minimum.	Maximum.		Average.	Minimum.	
Rivet-rods.	5/8	48000	57000	66.0	29.0	27.0	60
"	3/4	48000	56000	65.0	30.0	28.0	60
"	7/8	47000	56000	63.5	30.5	28.5	58
"	1	47000	56000	62.0	31.0	29.0	56
"	1 1/8	46000	56000	60.5	31.0	29.0	54
"	1 1/4	46000	56000	59.0	31.0	29.0	52

* Private communication, February, 1896.

Two tons of bars from the same heat shall constitute a lot, and two specimens, each from a different bar, shall be tested from each lot. The above table gives the average required of these two bars, and the minimum below which no bar shall fall. If the average elongation or reduction of area on any one lot shall fall below the requirement, two additional bars shall be cut from the same lot, and the average of the four pieces shall be considered the average of the lot, provided that no concession be made in the minimum. Heats rolled into sizes less than five-eighths inch may be tested in trial rods of three-quarters inch.

A rolled round about three-quarters inch in diameter, after being nicked one-quarter way through, shall bend completely double without fracture, with the nick on the outer curve of the bend. A piece of three-quarter-inch rod cut one-half inch long shall be upset while cold into a disk one-eighth inch thick, without developing extensive flaws or showing signs of cold-shortness.

Rivets, when cut out of the work into which they have been put, shall show a tough silky structure, with no crystalline appearance.

See also general provisions, p. 756.

CLASS III.—HARD BRIDGE RIVETS; A SUBSTITUTE FOR CLASS II, GIVING GREATER STRENGTH WITH LESS TOUGHNESS.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .04 in acid steel, below .03 in basic; S below .05; Si below .04; Mn below .60.

Physical requirements as follows:

Shape.	Diameter in Inches.	Ultimate Strength. Lbs. per Square Inch.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.		Average Reduction of Area, Per Cent.
		Minimum.	Maximum.		Average.	Minimum.	
Rivet-rods.	5/8	54000	63000	61.0	28.0	26.0	55
"	3/4	54000	62000	60.0	29.0	27.5	55
"	7/8	53000	62000	58.5	29.5	27.5	53
"	1	53000	62000	57.0	30 0	28.0	51
"	1 1/8	52000	62000	55.5	30.0	28.0	49
"	1 1/4	52000	62000	54 0	30.0	28.0	47

Two tons of bars from the same heat shall constitute a lot, and two specimens, each from a different bar, shall be tested from each lot. The above table gives the average required of these two bars, and the minimum below which no bar shall fall. If the average elongation or reduction of area on any one lot shall fall below the requirement, two additional bars shall be cut from the same lot, and the average of the four pieces shall be considered the average of the lot, provided that no concession be made in the minimum. Heats rolled into sizes less than five-eighths inch may be tested in trial rods of three-quarters inch.

Rivets, when cut out of the work into which they have been put, shall show a tough silky structure, with no crystalline appearance.

See also general provisions p. 756.

CLASS IV.—COMMON HARD RIVETS; FOR ROOF-TRUSSES AND OTHER STRUCTURES NOT EXPOSED TO SHOCK.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .05; Si below .04; Mn below .60.

Physical requirements as follows:

Shape.	Diameter in Inches.	Ultimate Strength, Pounds per Square Inch.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.
		Minimum.	Maximum.			
Rivet-rods.	5/8	54000	63000	61.0	27.0	55
"	3/4	54000	62000	60.0	28.0	55
"	7/8	53000	62000	58.5	28.5	53
"	1	53000	62000	57.0	29.0	51
"	1 1/8	52000	62000	55.5	29.0	49
"	1 1/4	52000	62000	54.0	29.0	47

Four tests shall be taken from each heat, and the average of these four shall conform to the above table. If the average elongation or reduction of area of any heat shall fall below the requirement, four additional bars may be cut from the same heat, and the average of the eight pieces shall be considered the average of the heat. Heats rolled into sizes less than five-eighths inch may be tested in trial rods of three-quarters inch.

Rivets, when cut out of the work into which they have been put, shall show a tough silky structure, with no crystalline appearance.

See also general provisions, p. 756.

CLASS V.—SOFT BRIDGE STEEL; FOR ANGLES, PLATES, BARS, ETC., FOR BRIDGES, CRANES, AND SIMILAR STRUCTURES EXPOSED TO SHOCK.

Method of manufacture, in per cent: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .07 in plates and angles, below .06 in eye-bars; Si below .04; Mn below .50.

Physical requirements as follows:

Shape.	Thickness in Inches.	Ultimate Strength, Lbs. per Sq. In.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.	Remarks.
		Minimum.	Maximum.				
Angles.	3/8	50000	58000	63.0	29.0	55	One piece of 3/4-inch angle must open out flat and another close shut without sign of fracture.
	1/2	50000	58000	61.5	29.0	53	
	5/8	49000	58000	60.0	29.0	51	
	3/4	49000	58000	58.5	29.0	49	
	7/8	48000	58000	57.0	29.0	47	
Plates.	5/16	53000	63000	65.0	28.0	44	On plates under 42 inches wide the required elongation shall be raised 1.5 per cent. and the reduction of area 2.0 per cent. On plates over 70 inches wide the elongation shall be lowered 1.5 per cent. and the reduction of area 2.0 per cent. On tests cut crosswise from the sheet, the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent. and the reduction of area 10 per cent. On universal mill-plates the allowance for transverse tests shall be 5000 lbs., 5 per cent and 15 per cent. Both longitudinal and transverse strips cut from plates shall bend double flat. When every plate in the heat is tested, the minimum elongation and reduction shall be lowered 5 per cent.
	3/8	51000	61000	63.0	26.0	50	
	1/2	50000	60000	62.0	26.0	50	
	3/4	49000	59000	60 0	25.0	48	
	1	48000	58000	58.0	24.0	46	
	1 1/4	47000	58000	56.0	23.0	44	
Eye-bars, annealed.	3/4	50000	58000	57.0	......	......	The elongation in full length shall be 15 per cent in bars from 10 to 20 ft. long, 14 per cent in 21 to 25 ft., 13.5 per cent in 26 to 30 ft., and 13 per cent in 31 to 35 ft.
	1	50000	58000	56.0	......	......	
	1 1/2	49000	58000	54.0	......	......	
	2	49000	58000	53.0	......	......	
	2 1/2	48000	58000	52.0	......	......	

SHAPES.—In channels, beams, etc., the requirements on tests cut from the web shall be the same as for plates between 42 and 70 inches wide, with the same allowance for difference in thickness. In tests cut from the flange the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent.

See also general provisions, p. 756.

CLASS VI.—MEDIUM BRIDGE STEEL; A SUBSTITUTE FOR CLASS V WHEN GREATER STRENGTH AND LESS TOUGHNESS ARE REQUIRED.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .07 in plates and angles, below .06 in eye-bars; Si below .04; Mn below .60.

Physical requirements as follows:

Shape.	Thickness in Inches.	Ultimate Strength, Lbs. per Sq. In. Minimum.	Ultimate Strength, Lbs. per Sq. In. Maximum.	Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.	Remarks.
Angles.	3/8	56000	64000	63.0	27.0	50	One piece of angle, not over 1/2 inch thick, shall open out flat, and another close shut without sign of fracture.
	1/2	56000	64000	61.5	27.0	48	
	5/8	55000	64000	60.0	27.0	46	
	3/4	55000	64000	58.5	27.0	44	
	7/8	54000	64000	57.0	27.0	42	
Plates.	5/16	59000	69000	62.0	22.0	39	On plates under 42 inches wide the required elongation shall be raised 1.5 per cent, and the reduction of area 2.0 per cent. On plates over 70 inches wide, the elongation shall be lowered 1.5 per cent, and the reduction of area 2.0 per cent. On tests cut crosswise from the sheet the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent. On universal mill-plates the allowance for transverse tests shall be 5000 lbs., 5 per cent and 15 per cent. Longitudinal strips shall bend double flat; transverse strips shall bend through 180 degrees around a pin 1 inch in diameter. When every plate in the heat is tested, the minimum elongation and reduction of area shall be lowered 5 per cent.
	3/8	57000	67000	60.0	25.0	45	
	1/2	56000	66000	59.0	25.0	45	
	3/4	55000	65000	57.0	24.0	43	
	1	54000	64000	55.0	23.0	41	
	1 1/4	53000	64000	53.0	22.0	39	
Eye-bars, annealed.	3/4	56000	64000	56.0	....	......	The elongation in full length shall be 14 per cent in bars from 10 to 20 ft. long, 13 per cent in 21 to 25 ft., 12.5 per cent in 26 to 30 ft., and 12 per cent in 31 to 35 ft.
	1	56000	64000	55.0	......	......	
	1 1/2	55000	64000	53.0	......	......	
	2	55000	64000	52.0	......	......	
	2 1/2	54000	64000	51.0	......	......	

SHAPES.—In channels, beams, etc., the requirements on tests cut from the web shall be the same as for plates between 42 and 70 inches wide, with the same allowance in thickness. In tests cut from the flange, the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent.

NOTE.—The allowable content of phosphorus may be raised to .08 per cent for acid and .05 per cent for basic steel, if the best quality is not required, but other specifications must remain the same.

See also general provisions, p. 756.

CLASS VII.—HARD BRIDGE STEEL.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .07 in plates and angles, below .06 in eye-bars; Si below .05; Mn below .80.

Physical requirements as follows:

Shape.	Thickness in Inches.	Ultimate Strength, Lbs. per Sq. In. Minimum.	Ultimate Strength, Lbs. per Sq. In. Maximum.	Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.	Remarks.
Angles.	3/8	60000	68000	62.0	26.0	48	One piece of angle, less than 1/2 inch thick, shall open out flat, and another piece close shut without sign of fracture.
	1/2	60000	68000	60.5	26.0	46	
	5/8	59000	68000	59.0	26.0	44	
	3/4	58000	68000	57.5	26.0	42	
	7/8	57000	68000	56.0	26.0	40	
Plates.	5/16	63000	73000	60.0	20.0	34	On plates under 42 inches wide the required elongation shall be raised 1.5 per cent, and the reduction of area 2.0 per cent. On plates over 70 inches wide, the elongation shall be lowered 1.5 per cent, and the reduction of area 2.0 per cent. On tests cut crosswise from the sheet the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent. On universal mill-plates the allowance for transverse tests shall be 5000 lbs., 5 per cent and 15 per cent. Longitudinal strips shall bend double flat. Transverse strips shall bend through 180 degrees around a pin 1 inch in diameter. When every plate in the heat is to be tested, the minimum elongation and reduction of area shall be lowered 5 per cent.
	3/8	61000	71000	58.0	23.0	40	
	1/2	60000	70000	57.0	23.0	40	
	3/4	59000	69000	55.0	22.0	38	
	1	58000	68000	53.0	21.0	36	
	1 1/4	57000	68000	51.0	20.0	34	
Eye-bars, annealed.	3/4	60000	68000	55.0	......	......	The elongation in full length shall be 13 per cent in bars from 10 to 20 ft. long, 12.5 per cent in 21 to 25 ft., 12 per cent in 26 to 30 ft., and 11.5 per cent in 31 to 35 ft.
	1	60000	68000	51.0	......	......	
	1 1/2	59000	68000	52.0	......	......	
	2	59000	68000	51.0	......	......	
	2 1/2	58000	68000	50.0	......	......	

SHAPES.—In channels, beams, etc., the requirements on tests cut from the web shall be the same as for plates between 42 and 70 inches wide, with the same allowance for difference in thickness. In tests cut from the flange the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent.

NOTE.—The allowable content of phosphorus may be raised to .08 per cent in acid and .05 per cent in basic steel, if the best quality is not required, but other specifications must remain the same.

See also general provisions, p. 756.

CLASS VIII.—EXTRA HARD BRIDGE STEEL; FOR SPECIAL STRUCTURES WHERE GREAT STIFFNESS IS ESSENTIAL.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .07 in plates and angles, below .06 in eye-bars; Si below .10; Mn below .80.

Physical requirements as follows:

Shape.	Thickness in Inches.	Ultimate Strength, Lbs. per Sq. In. Minimum.	Ultimate Strength, Lbs. per Sq. In. Maximum.	Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.	Remarks.
Angles.	3/8	64000	72000	61.0	25.0	45	One piece of angle, about 3/8-inch thick, shall open out flat, and another piece close shut without sign of fracture.
	1/2	64000	72000	59.5	25.0	43	
	5/8	63000	72000	58.0	25.0	41	
	3/4	62000	72000	56.5	25.0	39	
	7/8	61000	72000	55.0	25.0	37	
Plates.	5/16	67000	77000	59.0	18.0	32	On plates under 42 inches wide the required elongation shall be raised 1.5 per cent, and the reduction of area 2.0 per cent. On plates over 70 inches wide the elongation shall be lowered 1.5 per cent, and the reduction of area 2.0 per cent. On tests cut crosswise from the sheet the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent. On universal mill-plates the allowance for transverse tests shall be 5000 lbs., 5 per cent and 15 per cent. Longitudinal strips shall bend double flat. When every plate in the heat is to be tested, the minimum elongation and reduction of area shall be lowered 5 per cent.
	3/8	65000	75000	57.0	21.0	38	
	1/2	64000	74000	56.0	21.0	38	
	3/4	63000	73000	54.0	20.0	36	
	1	62000	72000	52.0	19.0	34	
	1¼	61000	72000	50.0	18.0	32	
Eye-bars, annealed.	3/4	64000	72000	54.0	......	......	The elongation in full length shall be 12.5 per cent in bars from 10 to 20 ft. long, 12.0 per cent in 21 to 25 ft., 11.5 per cent in 26 to 30 ft., and 11.0 per cent in 31 to 35 ft.
	1	64000	72000	53.0	......	......	
	1½	63000	72000	51.0	......	.....	
	2	63000	72000	50.0	......	......	
	2½	62000	72000	49.0	.....	.. ...	

SHAPES.—In channels, beams, etc., the requirements on tests cut from the web shall be the same as for plates between 42 and 70 inches wide, with the same allowances for difference in thickness. In tests cut from the flange the minimum tensile strength shall be lowered 3000 lbs., the elongation 3 per cent, and the reduction of area 10 per cent.

NOTE.—The allowable content of phosphorus may be raised to .08 per cent for acid steel and .05 per cent for basic, if the best quality is not required, but other specifications must remain the same.

See also general provisions, p. 756.

CLASS IX.—FORGING STEEL; FOR PINS AND MISCELLANEOUS FORGINGS AND FOR SPECIAL PLATES AND ANGLES.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .06 in acid steel, below .04 in basic; S below .07 in plates and angles, below .06 in eye-bars; Si below .10; Mn below .90.

Physical requirements as follows:

Shape.	Thickness in Inches.	Ultimate Strength, Lbs. per Sq. In.		Elastic Ratio.	Elongation in 8 Inches, Per Cent.	Reduction of Area, Per Cent.	Remarks.
		Minimum.	Maximum.				
Angles.	⅜	70000	80000	58.0	22.0	42	One piece of angle ⅝-inch thick, shall open out flat, and another piece close shut without sign of fracture.
	½	70000	80000	56.5	22.0	40	
	⅝	69000	80000	55.0	22.0	38	
	¾	68000	80000	53.5	22.0	36	
	⅞	67000	80000	52.0	22.0	34	
Plates.	5/16	73000	83000	56.0	16.0	80	On plates under 42 inches wide the required elongation shall be raised 1.5 per cent, and the reduction of area 2.0 per cent. On plates over 70 inches wide, the elongation shall be lowered 1.5 per cent, and the reduction of area 2.0 per cent. Longitudinal strips under ½ inch thick shall bend double flat. When every plate in the heat is to be tested, the minimum elongation and reduction of area shall be lowered 5 per cent.
	⅜	71000	81000	54.0	19.0	36	
	½	70000	80000	53.0	19.0	36	
	¾	69000	79000	51.0	18.0	34	
	1	68000	78000	49.0	17.0	32	
	1¼	67000	78000	47.0	16.0	30	

When this steel is used for pins or forgings, a charge may be tested by rolling a small test ingot or piece of bloom into a bar with a cross-section of about 0.5 or 1.0 square inch. This bar should have an ultimate strength of between 70,000 and 80,000 pounds per square inch, an elastic ratio of 58 per cent, and an elongation of 15 per cent in eight inches. This method will usually suffice to show the quality of the steel. If it is desirable to test the forged work, a bar should be cut from a rolled or hammered piece about six inches in smallest dimension, and turned to a three-quarter-inch round, two inches between shoulders. This should have an ultimate strength of between 67,000 and 80,000 pounds per square inch, an elastic ratio of 50 per cent, and elongation of 20 per cent in two inches. The test-piece should be cut at a depth of about two inches from the outside.

See also general provisions, p. 756.

CLASS X.—HARD FORGING STEEL; FOR MISCELLANEOUS FORGINGS.

Method of manufacture: Acid or basic open-hearth process.

Chemical composition, in per cent: P below .05 in acid steel, below .03 in basic; S below .07; S below .10: Mn below .90.

Physical requirements as follows:

Shape and Origin of Test-piece.	Ultimate Strength, Pounds per Square Inch. Minimum.	Ultimate Strength, Pounds per Square Inch. Maximum.	Elastic Ratio.	Elongation, Per Cent.
A rolled bar with a cross-section of about 0.5 to 1.0 square inch, made from a bloom or test ingot. Elongation measured in 8 inches........	75000	100000	55	18
A ¾-inch round, 2 inches long between shoulders, cut from a rolled or forged piece about 6 inches in smallest dimension. Elongation measured in 2 inches..	75000	100000	45	15

The first method will suffice for ordinary work to show the quality of the material. The second involves considerable expense and delay in cutting and finishing the piece, and there is necessarily much variation caused by the different sizes and shapes of forgings. The test-piece should be cut at a depth of about two inches from the outside.

See also general provisions, p. 756.

CLASSES XI, XII, AND XIII.—For buildings, highway bridges, and other structures not exposed to shock.

The requirements on material for ordinary structures need not be so carefully drawn as in the case of railroad bridges. Hence it will suffice to accept the standard specifications of the Association of American Steel Manufacturers given on the following pages. They are here given in full, since the clauses relating to the inspection of material, and the allowance for overweights, apply equally to all classes of material. The Association did not limit the use of this metal to buildings not exposed to shock, for the matter of chemical composition was left open to the engineer, but, in common with almost all manufacturers, I must unqualifiedly condemn the use of metal for a railway bridge that contains over .08 per cent of phosphorus, while I believe that .06 per cent should be the upper limit.

III.—STANDARD SPECIFICATIONS GOVERNING THE CHEMICAL AND PHYSICAL PROPERTIES OF STRUCTURAL AND SPECIAL OPEN-HEARTH PLATE AND RIVET STEEL, AS ADOPTED BY THE ASSOCIATION OF AMERICAN STEEL MANUFACTURERS* ON AUGUST 9TH, 1895, REVISED JULY 17TH, 1896,

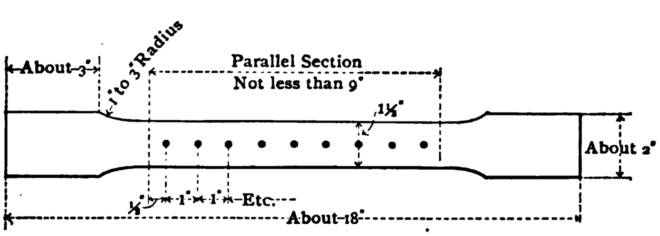
AND SINCE FORMALLY APPROVED BY THE FOLLOWING COMPANIES: THE BETHLEHEM IRON CO.; CAMBRIA IRON CO.; CARBON STEEL CO.; THE CARNEGIE STEEL CO., LIMITED; CATASAUQUA MANUFACTURING CO.; CENTRAL IRON WORKS; CLEVELAND ROLLING MILL CO.; COLORADO FUEL AND IRON CO.; GLASGOW IRON CO.; ILLINOIS STEEL CO.; JONES & LAUGHLINS, LIMITED; LUKENS IRON AND STEEL CO.; OTIS STEEL CO., LIMITED; PACIFIC ROLLING MILL CO.; PASSAIC ROLLING MILL CO.; PAXTON ROLLING MILLS; PENNSYLVANIA STEEL CO.; POTTSTOWN IRON CO.; POTTSVILLE IRON AND STEEL CO.; READING ROLLING MILL CO.; SHOENBERGER STEEL CO.; SPANG STEEL AND IRON CO.; WORTH BROS.

STRUCTURAL STEEL.

Process of Manufacture. 1. Steel may be made by either the Open-hearth or Bessemer process.

Testing. 2. All tests and inspections shall be made at place of manufacture prior to shipment.

Test-pieces. 3. The tensile strength, limit of elasticity, and ductility shall be determined from a standard test-piece cut from the finished material. The standard shape of the test-piece for sheared plates shall be as shown by the following sketch:



Piece to be of same thickness as the plate.

On tests cut from other material the test-piece may be either the same as for plates, or it may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of 8 inches, except when the thickness of the finished material 5/16 inch or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and except in rounds of 5/8 inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Two test-pieces shall be taken from each melt or blow of finished material, one for tension and one for bending.

Annealed Test-pieces. 4. Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

* These specifications, as amended from year to year by the manufacturers themselves, are likely to come into very general use, since any deviation from them will involve additional expense.—J. B. J.

Marking. 5. Every finished piece of steel shall be stamped with the blow or melt number, and steel for pins shall have the blow or melt number stamped on the ends. Rivet and lacing steel and small pieces for pin-plates and stiffeners may be shipped in bundles securely wired together, with the blow or melt number on a metal tag attached.

Finish. 6. Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish.

Chemical Properties. 7.

Steel for Railway Bridges:	Maximum Phosphorus .08 per cent.
Steel for Buildings, Train Sheds, Highway Bridges, and similar structures:	Maximum Phosphorus .10 per cent.

Physical Properties. 8. Steel shall be of three grades, RIVET, SOFT, and MEDIUM.

Rivet Steel. 9. Ultimate strength, 48,000 to 58,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 26 per cent.
Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

Soft Steel. 10. Ultimate strength, 52,000 to 62,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 25 per cent.
Bending test, 180 degrees flat on itself, without fracture on outside of bent portion.

Medium Steel. 11. Ultimate strength, 60,000 to 70,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 22 per cent.
Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

Pin Steel. 12. Pins made from either of the above-mentioned grades of steel shall, on specimen test-pieces cut at a depth of one inch from surface of finished material, fill the physical requirements of the grade of steel from which they are rolled, for ultimate strength, elastic limit, and bending, but the required elongation shall be decreased 5 per cent.

Eye-bar Steel. 13. Eye-bar material, 1½ inches and less in thickness, made of either of the above-mentioned grades of steel, shall, on test-pieces cut from finished material, fill the requirements of the grade of steel from which it is rolled. For thicknesses greater than 1½ inches there will be allowed a reduction in the percentage of elongation of 1 per cent for each 1/8 of an inch increase of thickness, to a minimum of 20 per cent for medium steel and 22 per cent for soft steel.

Full-size Test of Steel Eye-bars. 14. Full-size test of steel eye-bars shall be required to show not less than 10 per cent elongation in the body of the bar, and tensile strength not more than 5000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which they are rolled. The bars will be required to break in the body; but should a bar break in the head, but develop 10 per cent elongation and the ultimate strength specified, it shall not be cause for rejection, provided not more than one third of the total number of bars tested break in the head; otherwise the entire lot will be rejected.

Variation in Weight. 15. The variation in cross-section or weight of more than 2½ per cent from that specified will be sufficient cause for rejection except in the case of sheared plates, which will be covered by the following permissible variations:

a. Plates 12½ lbs. or heavier, when ordered to weight, shall not average more variation than 2½ per cent, either above or below the theoretical weight.

b. Plates from 10 to 12½ lbs., when ordered to weight, shall not average a greater variation than the following:

Up to 75 inches wide, 2½ per cent either above or below the theoretical weight.

75 inches and over, 5 per cent, either above or below the theoretical weight.

c. For all plates ordered to gauge there will be permitted an average excess of weight over that corresponding to the dimensions on the order equal in amount to that specified in the following table:

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

The weight of 1 cubic inch of rolled steel is assumed to be .2833 pound.

Plates 1/4" and over in Thickness.				Plates under 1/4" in Thickness.		
Thickness of Plate.	Width of Plate.			Thickness of Plate.	Width of Plate.	
	Up to 75 in.	75 to 100 in.	Over 100 in.		Up to 50 in.	50 in. and above.
1/4 in.	10 per cent.	14 per cent.	18 per cent.	1/8 up to 5/32	10 per cent.	15 per cent.
5/16 "	8 "	12 "	16 "	5/32 " 3/16	8½ "	12½ "
3/8 "	7 "	10 "	13 "	3/16 " 1/4	7 "	10 "
7/16 "	6 "	8 "	10 "			
1/2 "	5 "	7 "	9 "			
9/16 "	4½ "	6½ "	8½ "			
5/8 "	4 "	6 "	8 "			
Over 5/8 "	3½ "	5 "	6½ "			

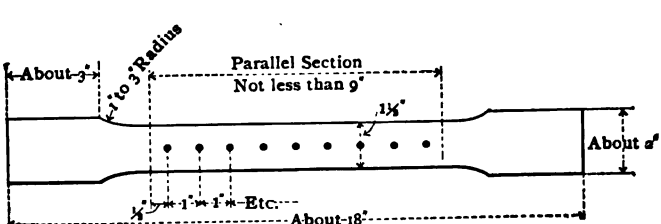
STRUCTURAL CAST IRON.

1. Except where chilled iron is specified, all castings shall be tough gray iron, free from injurious cold-shuts or blow-holes, true to pattern, and of a workmanlike finish. Sample pieces, one inch square, cast from the same heat of metal in sand-moulds, shall be capable of sustaining on a clear span of 4 feet 8 inches a central load of 500 pounds when tested in the rough bar

SPECIAL OPEN-HEARTH PLATE AND RIVET STEEL.

Testing and Inspection. 1. All tests and inspections shall be made at place of manufacture prior to shipment.

Test-pieces. 2. The tensile strength, limit of elasticity, and ductility shall be determined from a standard test-piece cut from the finished material. The standard shape of the test-piece for sheared plates shall be as shown by the following sketch:



Piece to be of same thickness as the plate.

On tests cut from other material the test-piece may be either the same as for plates, or it may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of 8 inches, except when the thickness of the finished material is 5/16 inch or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and except in rounds of 5/8 inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Four test-pieces shall be taken from each melt of finished material; two for tension and two for bending.

Annealed Test-pieces. 3. Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

Marking. 4. Every finished piece of steel shall be stamped with the melt number. Rivet steel may be shipped in bundles securely wired together, with the melt number on a metal tag attached.

Finish. 5. All plates shall be free from surface defects and have a workmanlike finish.

Chemical Properties. 6.

Extra Soft and Fire-box Steel:	Maximum	Phosphorus	.04	per cent.
	"	Sulphur	.04	" "
Flange or Boiler Steel:	"	Phosphorus	.06	" "
	"	Sulphur	.04	" "
Boiler-rivet Steel:	"	Phosphorus	.04	" "
	"	Sulphur	.04	" "

Physical Properties. 7. Steel shall be of four grades—EXTRA SOFT, FIRE-BOX, FLANGE or BOILER, and BOILER-RIVET STEEL.

Extra Soft Steel. 8. Ultimate strength, 45,000 to 55,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 28 per cent.
Cold and Quench bends, 180 degrees flat on itself, without fracture on outside of bent portion.

Fire-box Steel. 9. Ultimate strength, 52,000 to 62,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 26 per cent.
Cold and Quench bends, 180 degrees flat on itself, without fracture on outside of bent portion.

Flange or Boiler-steel. 10. Ultimate strength, 52,000 to 62,000 pounds per square inch.
Elastic limit, not less than one half the ultimate strength.
Elongation, 25 per cent.
Cold and Quench bends, 180 degrees flat on itself, without fracture on outside of bent portion.

Boiler-rivet Steel. 11. Steel for boiler-rivets shall be made of the extra soft quality specified in paragraph No. 8.

Variation when ordered to Gauge. 12. For all plates ordered to gauge there will be permitted an average excess of weight over that corresponding to the dimensions on the order equal in amount to that specified in the following table, provided no plate shall be rejected for light gauge measuring .01″ or less below the ordered thickness:

TABLE OF ALLOWANCES FOR OVERWEIGHT FOR RECTANGULAR PLATES WHEN ORDERED TO GAUGE.

The weight of 1 cubic inch of rolled steel is assumed to be .2833 lb.

PLATES ¼″ AND OVER IN THICKNESS.

Thickness of Plate.	Width of Plate.		
	Up to 75 in.	75 in. to 100 in.	Over 100 in.
1/4 inch	10 per cent	14 per cent	18 per cent
5/16 "	8 " "	12 " "	16 " "
3/8 "	7 " "	10 " "	13 " "
7/16 "	6 " "	8 " "	10 " "
1/2 "	5 " "	7 " "	9 " "
9/16 "	4½ " "	6½ " "	8½ " "
5/8 "	4 " "	6 " "	8 " "
Over 5/8 "	3½ " "	5 " "	6½ " "

PLATES UNDER ¼″ IN THICKNESS.

Thickness of Plate.	Width of Plate.	
	Up to 50 in.	5o in. and above.
1/8 up to 5/32	10 per cent	15 per cent
5/32 " 3/16	8½ " "	12½ " "
3/16 " 1/4	7 " "	10 " "

Variation when ordered to Weight. 13. Plates 12½ lbs. or heavier, when ordered to weight, shall not average more variation than 2½ per cent, either above or below the theoretical weight.

Plates from 10 to 12½ lbs., when ordered to weight, shall not average a greater variation than the following:

Up to 75 inches wide, 2½ per cent either above or below the theo-theoretical weight.

75 inches and over, 5 per cent either above or below the theoretical weight.

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